SEMINAR ON

WATER SUPPLY IN PLANTATIONS

25,26th JULY, 1980 AT ST. JOHN'S MEDICAL COLLEGE,
BANGALORE.

TECHNICAL NOTES ON WATER RESOURCES
(EXCLUDING QUALITY AND HEALTH ASPECTS)

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AFPRO (Action For Food Production) was requested by St. John's Medical College on behalf of UPASI, (United Planter's Association of Southern India), to deal with the Resource aspects of Water Supply In Plantations in this Seminar.

A visit to selected plantations enabled crystallization of the problems, specially drinking water aspects were to be dealt with as it was felt that irrigation at this juncture is not a priority.

The notes have been prepared in as simple a form as possible and are relevant specifically to plantations in Southern India. It is sincerely hoped that these will be useful to Plantation Managers and Health officers for whose benefit this Seminar has been organised.

We gratefully acknoweldge the co-operation and assistance rendered by UPASI and by the various plantations visited. The information supplied by them have greatly inspired us to prepare the Seminar notes.

For AFPRO

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SESSION II

ASSESSMENT OF WATER RESOURCES IN PLANTATIONS

be thoroughly determined taking into consideration cost and availability of water, waste disposal problems, management and types of processes involved and requirements of the factory personnel which may be taken as 45 or 30 litres per head per day depending on whether bathing rooms are provided or not. The following table shows approximately the requirement of some industries:

Industry	Unit of Production	Water Requirements (Kilolitres/day) per unit of production
Automobile	Vehicle	40
Distillery	Kilolitres	122-170
Fertilizer	Tonne	80-200
Leather	100 kg.(tanned)	4
Paper	Tonne	200-400
Strawboard	"	75-100
Petroleum Refinery	" (crude)	1.5-2.0
Steel	"	200-250
Sugar	" (cane crushed)	1-2
Textile	100 kg.(goods)	8-14

The water requirements for hospitals (including laundering) is 455 litres per bed per day in case the number of beds is over 100. For those with less than 100 beds, the daily requirement reduces to 340 litres per bed per day.

The daily requirement for livestock varies from 7.5 litres for a sheep to 132 litres for a dairy cow. Incidently, plantations do not have large scale livestock development and hence actual requirement may not be appreciable as compared to requirement for other uses.

The daily water requirement for a typical estate of 2,500 acres with a population of 2,000 a rubber factory, a hospital with 50 beds and 50 head of milch cattle is shown below (in litres per day)

..3/-

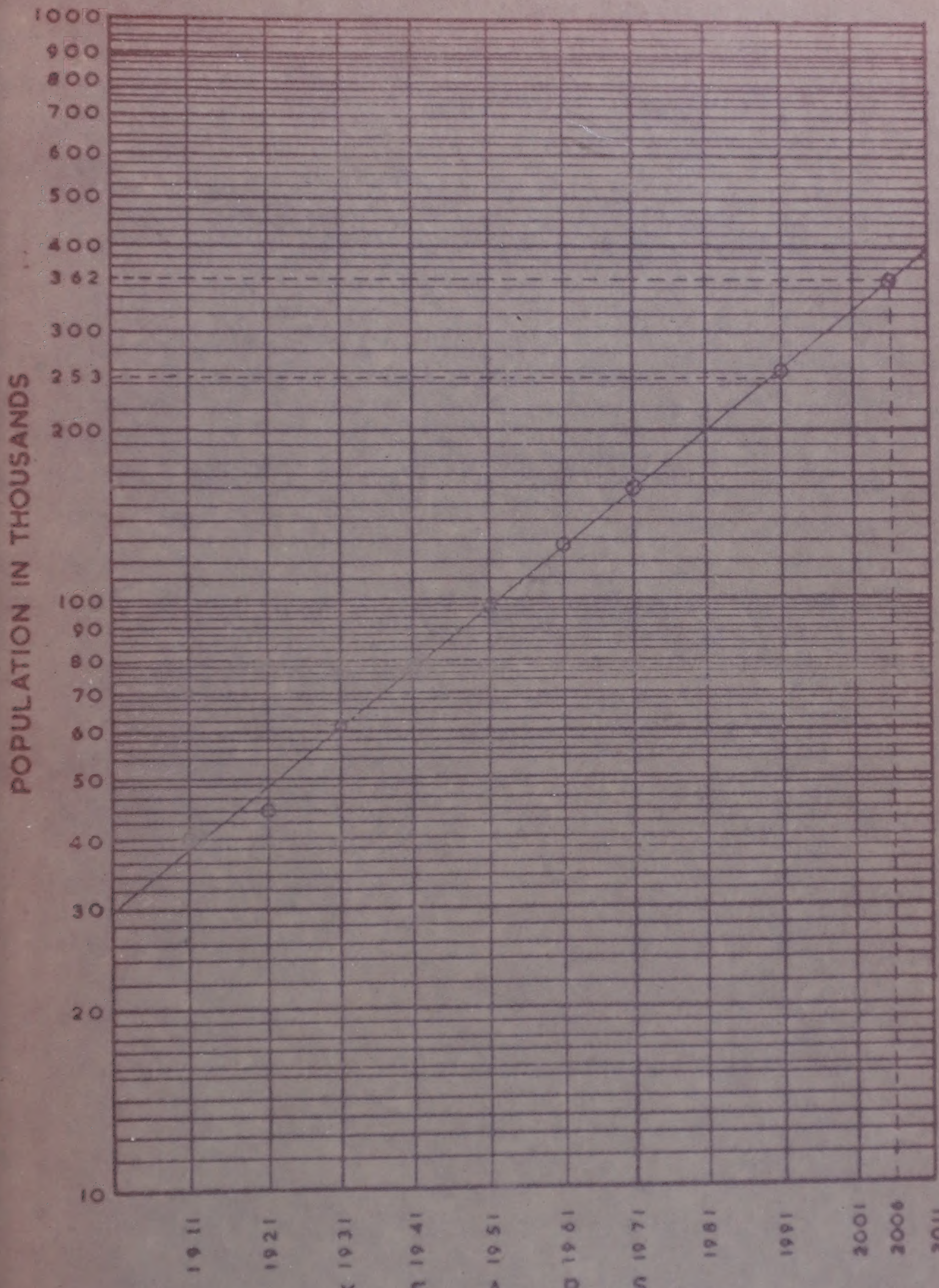


FIG. I SEMI LOG GRAPH FOR ESTIMATION OF FUTURE POPULATION

Domestic @ 40 litres per head	80,000.00
Factory (approx.)	35,000.00
Hospital @ 340 litres per bed	17,000.00
Livestock @ 132 litres per head	6,600.00
Add 160 litres per head for 50 managerial staff	8,000.00
Total	1,46,000.00

The future demand is also an important criteria for the estimation of the requirements of an estate. The important variable here is the population and whilst making plans for future demands, a period of 20 years is considered adequate to project this. In figure 1 is shown a projection of future population on the basis of which future demands may be computed. The past records on population would be needed for this and hence such an attempt has not been made for the above typical estate.

Sources of Water In Plantations:

Before considering the sources of water that can be tapped in plantations it is imperative to touch on the earth's water cycle, or the 'Hydrologic cycle '. The ultimate source of all water resources is precipitation either from rainfall or snowfall or both, in case of the plantations the only precipitation is rainfall. Rainwater that falls on the earth goes into the ground as infiltration and percolation, over the surface as runoff and through the root zone via the vegetation back to the atmosphere as evapotranspiration. The water that percolates to deeper zones in the ground adds to the underground water reservoir. This groundwater is also dynamic with movement to lower regions and ultimately into the sea. Evaporation and evapotranspiration (through plants) takes the water back into the atmosphere thus completing the cycle, which is shown in figure 2. The plantation areas would constitute the upper parts of the topographic setting in the cycle where surface runoff, evaporation and transpiration are appreciable being near the topographical surface divides and covered with plantation crops and forests. The infiltration into the soil zone may also be appreciable but percolation into deeper zones is expected to be much less. Thus it can be appreciated that the source in the plantations would be primarily surface water and to a limited extent groundwater mostly in the topographical lows.

Surface water from perennial rivers, streams, brooks, etc., constitute the most important aspect of water resources in plantations. Groundwater resources are limited and open wells tapping the shallow soil zone and some part of the vadose zone are preferentially sunk near the rivers or streams. Borewell prospects are minimal and in fact have not been considered in most plantations. Such borewells though of a low discharge could serve partly the small scatterred communities. Another important source of water supplies are springs which are discharges of

Figure 2. Schematic diagram of the earth's water cycle — the hydrologic cycle.

ground water at the surface when travelling along cracks, crevices, joints or faults that outcrops at the surface. This is essentially groundwater that contributes to or becomes part of the surface discharge.

Surface water may be exploited by construction of weirs, small dams, etc., across rivers or streams or simply by sinking jack or collector wells in the river or stream bed. Perennial rivers with voluminous discharges need no structures to impound water. Ground water may be extracted by sinking shallow open wells in the topographical lows, borewells along areas with suspected fracture zones or deep weathered zones in rocks and by tapping of supplies from springs.

Investigation for and Estimation of Groundwater:

Groundwater is obtained through aquifers-those parts of a rock formation which are capable of transmitting and yielding viable quantities of water. Rocks may be consolidated,(compacted, cemented eg., Granite,sandstone,limestone), or unconsolidated (loose materials eg., clay,sand,gravel), rocks. These may be termed hard and soft rocks respectively. Further rocks maybe classified based on their origin as:

Sedimentary: Those formed by deposit of materials derived from weathering and corrosion of other rocks. These constitute only about 5% of the earth's crust but contain an estimated 95% of available ground water. These rocks maybe unconsolidated eg., sand,gravel or mixtures of both, and are the best aquifers available. Others include marine deposits, alluvial or river/ stream deposits, glacial, wind and drift blown sand and loess. Sandstone is an example of a consolidated sedimentary rock.

Igneous: Those resulting from the consolidation of hot molten magma, which originates at great depths in the earth. If solidification occurs at depth they give rise to intrusive or plutonic rock, eg., granites. Those solidifying at the surface are called extrusive or volcanic rocks- eg., basalts. While granites being coarse and non-porous, are not good aquifers, groundwater maybe expected in the crevices and fractures as also in the upper weathered portions. Basalts can be permeable and porous as a result of interconnected openings or vesciles formed when gas escapes whilst cooling. They may also contain groundwater in crevices, and in breciated tops and bottoms of successive lava flows.

Metamorphic: Those rocks either igneous or sedimentary which have been altered by heat and pressure eg., quartzite, slate and mica schists, gneiss which have formed by metamorphosis of sand-

stone, shale and granite respectively. Marble (metamorphosed limestone) maybe a good aquifer when fractured or containing solution channels.

The level to which groundwater rises and stands below ground level under atmospheric conditions is known as the 'Water Table'. Since all Plantations in South India lie on crystalline granitic or complex gneissic rocks we shall restrict ourselves to the occurence, investigation and assessment of groundwater in such terrains only. The predominant rocks are granites and gneisses neither of which contain interconnected pore spaces. Nearer the surface however, these maybe weathered and connected by intersecting joints and crevices. Such zones form valuable receptacles for groundwater. The lateral and vertical extent vary widely from place to place but in the plantations this is expected to be very shallow (20-30 feet or 7-10 metres) when compared to the plains.

Methods of Investigation for Groundwater:

As it is not entirely relevant to go into the details of the theory and field procedures of the various methods, we shall deal only with the applicability of these methods.

Hydrogeological: Involves study of the geology, structure, weathering pattern, joints, fractures and data related to the occurence of groundwater from existing wells and surface studies. This would be indicative of what may be expected if new wells have to be sunk and thus aid in proper design and construction.

Geophysical: By far the most useful of the four geophysical methods (electric, magnetic, seismic and gravimetric) is the electric method whereby the resistivity of the formations are determined; these vary for different rocks depending on the material, density, degree of weathering, porosity, pore size and shape, water content and quality and temperature. These are indirect methods to guide one as to the type and depth of

a well to work out prospects of encountering water bearing formations.

Hydrological: Involves collection and processing data on rainfall, percolation, water level fluctuations in wells, groundwater quality, determination of well yields, and water budgeting. This would enable us to work out the type and size of wells, quantity that should be pumped, any treatment required and work out a simple draft from the groundwater reservoir.

Thus it maybe seen that the various methods when applied enables one to determine the quantity of groundwater available, the exact location of wells, the depth to which such wells have to be sunk and the quantity that maybe tapped by each so as to maintain a balance between input and draft.

Estimation of Groundwater Availability:

Calculation of the groundwater budget is done usually on a large scale eg., basinwise. For purposes of illustration however, the groundwater budget of a typical estate may be calculated from the formula:

$$G = (P + I) - (ET + R)$$

where G = recharge/storage or available groundwater,

P = Precipitation or rainfall

I = Imported water (canals, surface water bodies, etc.,)

ET = Evapotranspiration losses;

and R = runoff

Thus assuming that an estate with an area of 2,500 acres, receiving annually 100 inches of rainfall looses 99% through evapotranspiration and runoff, but receives little or no recharge from other sources, the annual groundwater recharge may be calculated as:

$G = (100 + 0) - (99)$

$= 1"$ over 2,500 acres

$= 2,500$ acre - inch

= 256913250 litres
say 25.69 X 10^7 litres

Allowing for deficient rainfall years and unequal distribution, 75% of this may be available for utilization. Thus 19.27 X 10^7 litres may be available annually or about 5,00,000 litres maybe available daily. However, an important point to be remembered is that much of this is expected to contribute to the surface discharge through the year, thus maintaining a perennial surface flow as is evidenced from actual (qualitative) observations. The need to quantify this by guaging (measuring) the surface discharges hence becomes apparent.

On estimating the total usable groundwater at macro levels, (plantation), wells may be sited at appropriate points after proper investigations. The discharge or yield of each well that can be safely drawn must now be determined. In plantations, groundwater sources cannot be expected to yield large supplies but only supplement those from surface sources. Since borewells have not been sunk in most or perhaps any estate, the actual assessment of these has not been made. The wells tap mostly water table aquifers as opposed to confined aquifers(see figure 3). The water here flows under gravity at atmospheric pressure. The assessment of the exact capacity and more precisely the pumpage rate to be fixed for open wells may be carried out as discussed below:

Determination of Well Yields:

The testing of water wells is carried out to obtain information about the performance and efficiency of the well, reported in terms of yield, draw-down and specific capacity. This helps in selection of a proper pump thus reducing recurring costs and use of power. The second objective is to provide data from which the principle factors of aquifer performance can be calculated and is hence called an 'aquifer test'. Some import - ant definitions relating to pump testing of wells are given below: (Also refer to figure 4)

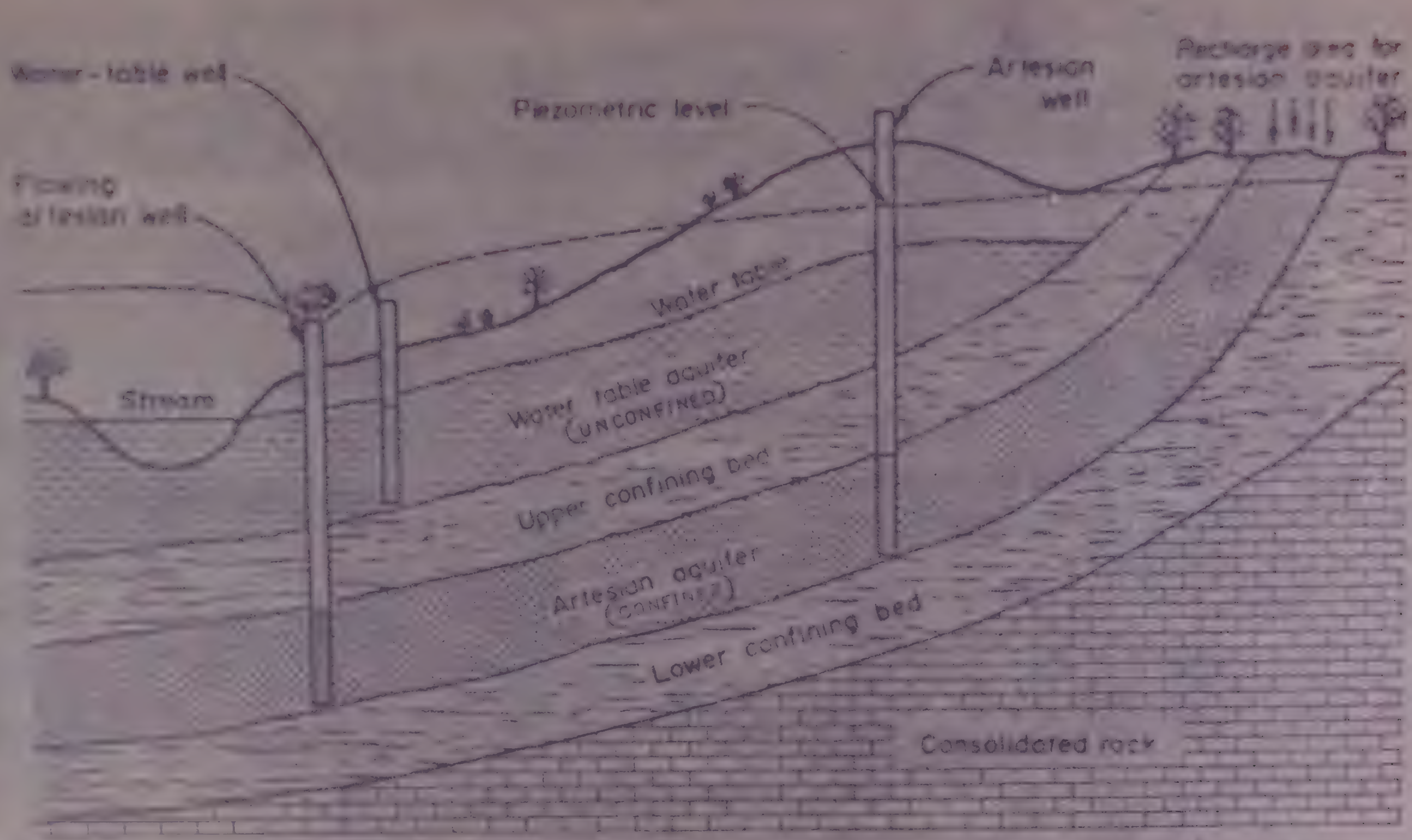


Figure 3. Subsurface and ground-water phase of the hydrologic cycle.

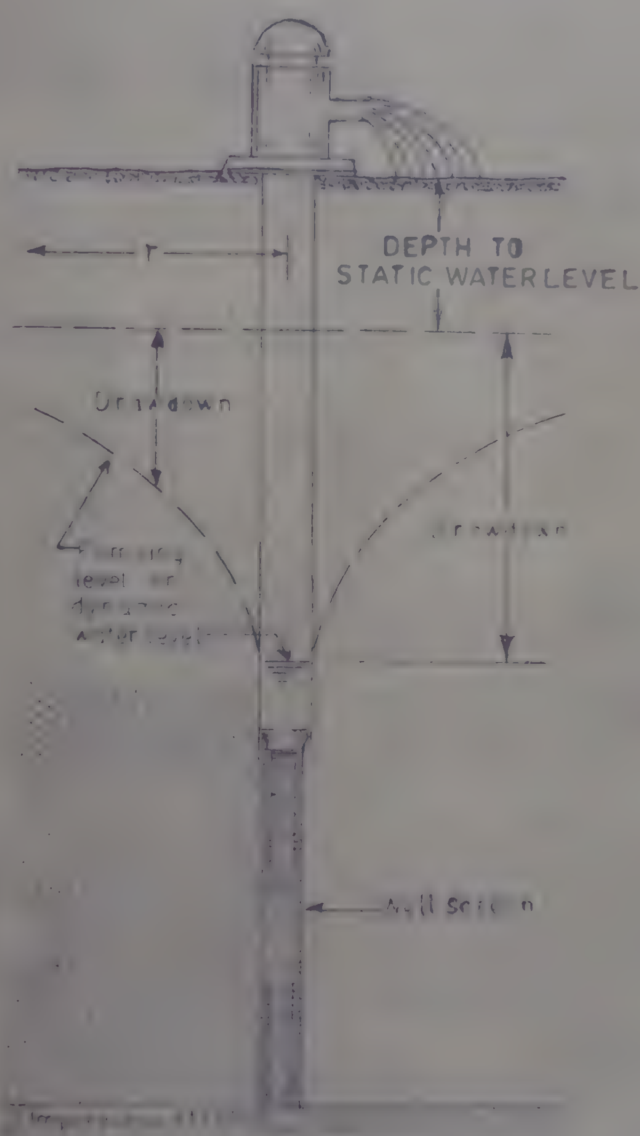


Fig. 4

Measurements related to well performance and pumping tests of wells and aquifers

Static Water Level(SWL): is the level at which water stands in a well when no water is drawn from the well either by pumping or free flow, and is expressed as distance from ground level to the water level.

Pumping Water Level (PWL): is the level at which water stands when pumping is in progress. In case of a flowing well it may be the level at which water is flowing from the well. This PWL may also be referred to as 'Dynamic Water Level'.

Drawdown(H): is the difference between the SWL and the PWL at any instant and represents the head in feet of water that causes water to flow through the aquifer material at the rate at which the water is being taken out.

Residual Drawdown: is the distance to the water level during the period when pumping stops and when the water level begins to recover to the SWL.

Well Yield: is the volume of water per unit time discharged from a well by pumping or by free flow, and is measured as the pumping rate in gallons per minute (gpm): gallons per hour(gph): for small yields or cubic feet per second (cfs) for large yields (Also in lpm, lph or m^3ps)

Specific Capacity: is the yield per unit draw down expressed as gpm per ft. drawdown (or lpm/metre drawdown) eg., at any given time if the pumping rate is 400 lpm and drawdown is 2 metres, the specific capacity may be expressed as 200 lpm per metre drawdown at the time measurements were taken.

Radius of Influence(R): is the distance from the centre of the well to the limit of the cone of depression(refer figure 4)

Coefficient of Storage (S): of an aquifer is the volume of water released or taken into storage per unit of surface area of the aquifer per unit change in head.

Coefficient of Transmissibility(T): of an aquifer is the rate at

which water will flow through a vertical strip of the aquifer of unit width extending the full thickness of the aquifer when the hydraulic gradient is 1 or 100%.

Coefficient of Permeability (P): of an aquifer is the rate at which water will flow through a Unit cross sectional area of the aquifer when the hydraulic gradient is 1 or 100% (T = Aquifer-thickness X P)

As has been explained earlier, the tests may be called Aquifer Test or a Well Test. The Aquifer Test consists of pumping and measuring water from a well and recording drawdown in the pumped well as well as in observation wells nearby (see figure 5). These data can show the characteristics of the aquifer. The measure-ments to be made for either test include SWL, pumping rate or discharge(Q in lpm), PWLs at various intervals, time of starting and stoppingof the pump, and water levels at intervals during recovery.

Measurements of Pump discharge:

A valve in a suitable sized discharge line of the pump provides the best control for various stages of pumping; the valve will be half to three fourths open at stages. A simple method for determining the pumping rate is to observe the timerequired to fill a container (say 200 litre barrel). If it takes 40 seconds to fill, the rate is 300 lpm. Such a method is practical when measuring small pumping rates.

Water Meter: This may be employed and is easy to use but causes delays in getting initial values at the start of a test when the rate is being adjusted to a desired level.

Circular Orifice: is the most commonly used device for measuring pumping rates and does not measure the pulsating flow from a pump. Figure 6 shows essential details of construction and assembly. The orifice is a perfectly round hole in the centre

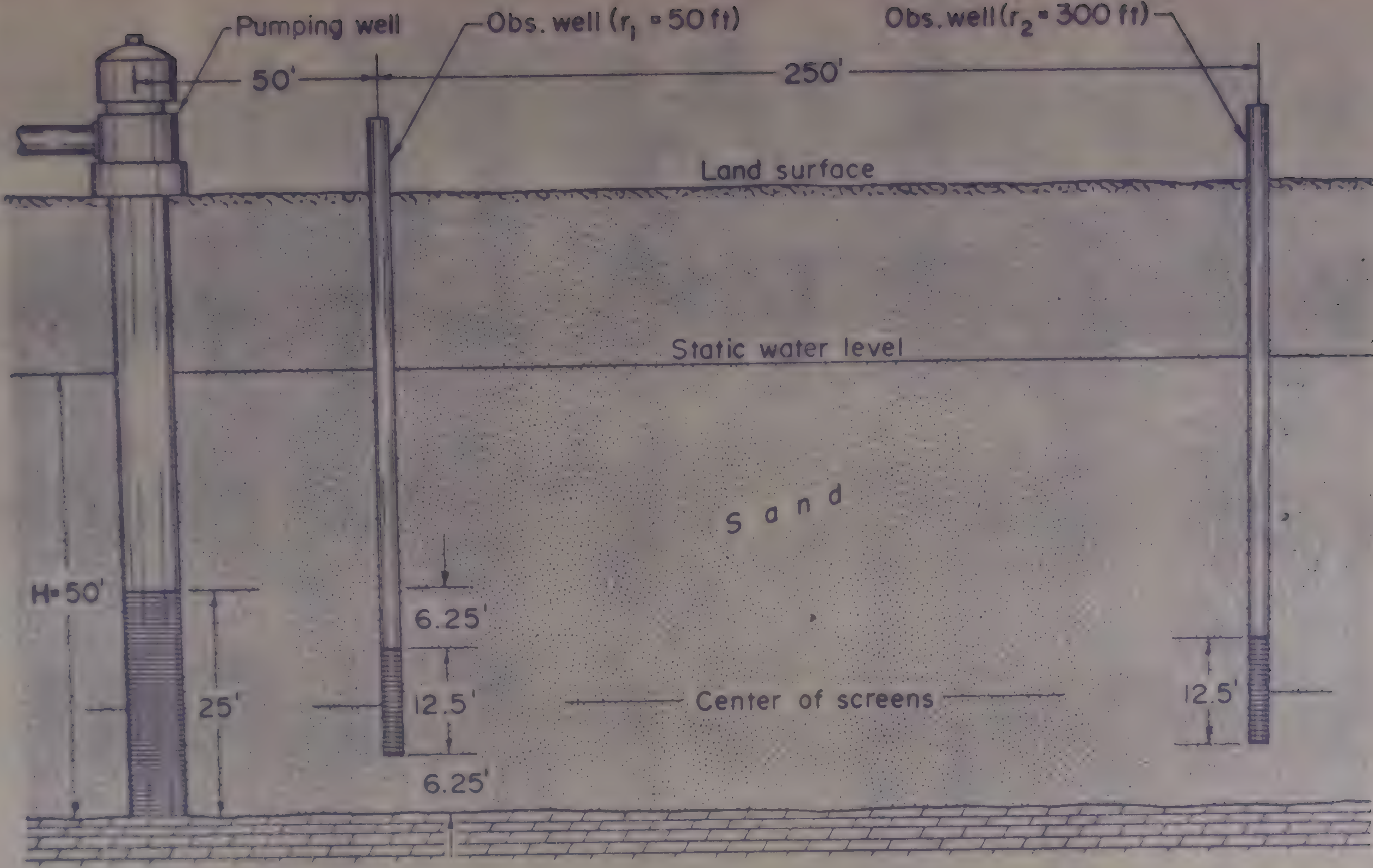
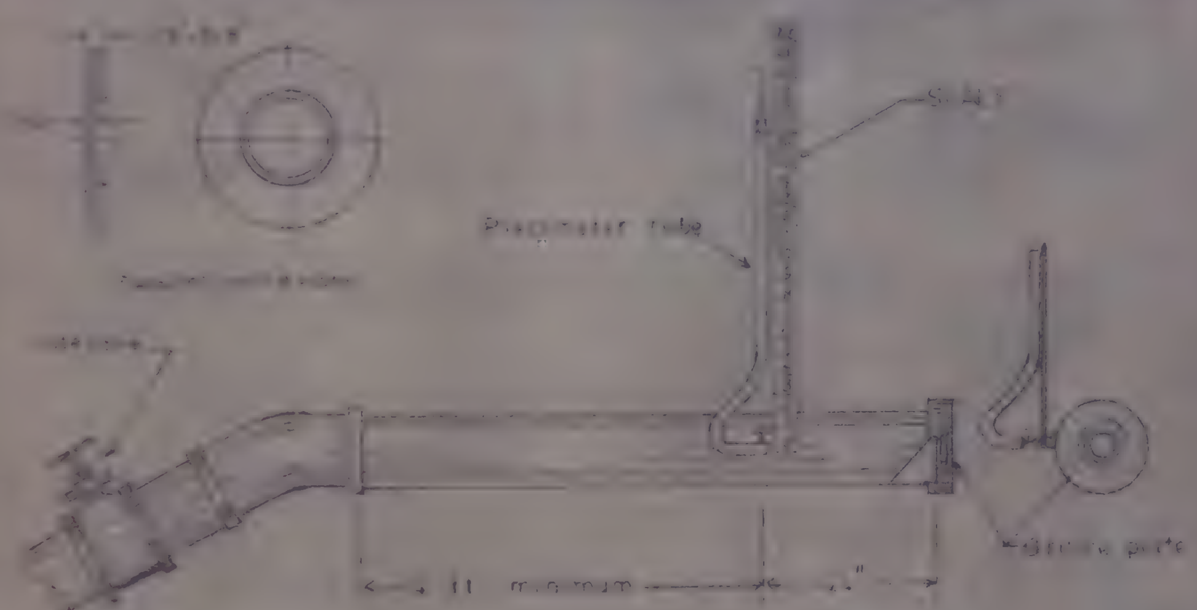


Figure 8. Typical arrangement of pumped well and observation wells for obtaining field data required to calculate coefficient of permeability from well discharge formulas.



6 Essential details of the circular orifice weir commonly used for measuring pumping rates when pumping by means of a turbine pump.
Discharge pipe must be level.

of a circular GI plate, 1/16" thick around the hole circumference. This is fastened square on the end of the 6 feet level discharge pipe and centred on the pipe, the bore of which must be smooth and free of obstruction from turbulence. Exactly 24" from the orifice plate the pipe wall is tapped midway by a 1/8" to 1/4" hole fitted with a nipple; a small plastic tube 4 or 5 feet long is fitted to this with a piece of glass tube at the other end. The nipple screwed into the tapped hole should not protrude inside the pipe. The tube called the piezometer tube is placed vertical against an accurate scale which is fastened absolutely vertical against the pipe. When water is pumped through the orifice the level in the piezometer represents pressure of water on the orifice. For any given orifice size and discharge pipe the rate of flow varies with pressure head. Standard tables are published giving flow for various combinations of orifice diameter and pipe diameter as shown in Table 1 (a) and (b). The flow through an orifice may be calculated from : $Q = 8.02\ K\ A\ \sqrt{h}$

where Q = flow per unit of time, gpm
A = area of orifice in square inches
h = head over orifice shown on piezometer tube, in inches
and K = Discharge factor which may be obtained from graph in figure 7

The precautions to be taken are:

(1) the diameter of the orifice should be less than 0.8 inches of the inner diameter of the pipe.
(2) the piezometer tube must be free from any obstruction and air bubbles.
(3) the control valve should be placed correctly as shown in figure 6, and as a practice should be away from the piezometer tube at a distance of at least 10 times the diameter of the pipe

Vertical and Horizontal pipe flow:

Figure 8 shows the arrangement to be made for measuring of flows

ORIFICE TABLES

For measurement of water through pipe orifices with free discharge.

The following tables have been compiled by the Engineering Department of Layne and Bowler, Incorporated, from original calibrations by Purdue University.

Head in Inches	3" Orifice 4 in. Pipe	3" Orifice 6 in. Pipe	4" Orifice 6 in. Pipe	4" Orifice 8 in. Pipe	5" Orifice 6 in. Pipe	5" Orifice 8 in. Pipe	6" Orifice 8 in. Pipe	6" Orifice 10 in. Pipe	7" Orifice 10 in. Pipe	8" Orifice 10 in. Pipe	Head in Inches
5	100	76	145	140	280	220	380	320			5
5.5	104	79	153	145	293	230	394	333			5.5
6	108	82	160	150	305	240	408	345			6
6.5	111	85	167	155	316	250	421	358			6.5
7	115	88	172	160	328	260	433	370			7
7.5	119	91	179	165	339	270	446	381			7.5
8	122	94	185	170	350	280	458	395	[illegible]	935	8
8.5	125	96	190	175	361	289	471	408	[illegible]	963	8.5
9	128	99	195	180	372	298	483	420	[illegible]	992	9
9.5	130	102	200	185	[illegible]	[illegible]	495	433	[illegible]	1016	9.5
10	133	104	205	190	[illegible]	[illegible]	509	441	[illegible]	1040	10
10.5	137	107	210	195	402	324	521	458	[illegible]	1060	10.5
11	140	109	218	200	412	330	533	470	[illegible]	1080	11
11.5	143	111	220	204	421	338	545	480	713	1100	11.5
12	146	114	225	208	430	346	556	[illegible]	728	1120	12
12.5	149	116	230	212	439	354	567	500	743	1139	12.5
13	151	118	234	216	448	362	[illegible]	510	[illegible]	1158	13
13.5	154	121	239	[illegible]	457	[illegible]	[illegible]	[illegible]	771	1176	13.5
14	157	123	243	224	465	376	[illegible]	[illegible]	[illegible]	1194	14
14.5	159	126	247	227	473	383	[illegible]	540	[illegible]	1212	14.5
15	162	128	[illegible]	231	[illegible]	390	[illegible]	[illegible]	[illegible]	[illegible]	15
15.5	164	130	254	234	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	15.5
16	167	132	257	238	495	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	16
16.5	170	134	261	241	503	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	16.5
17	172	136	264	245	510	414	[illegible]	[illegible]	[illegible]	[illegible]	17
17.5	175	138	268	249	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	17.5
18	178	140	271	252	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	18
18.5	180	142	275	256	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	18.5
19	183	144	278	259	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	19
19.5	185	146	282	[illegible]	542	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	19.5
20	187	148	285	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	20
20.5	190	150	[illegible]	270	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	20.5
21	193	152	292	273	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21
21.5	195	154	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21.5
22	197	156	[illegible]	279	[illegible]	[illegible]	744	[illegible]	[illegible]	[illegible]	22
22.5	199	158	[illegible]	282	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	22.5
23	201	[illegible]	[illegible]	285	584	479	760	678	1001	1500	23
23.5	203	[illegible]	307	288	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	23.5
24	205	164	310	291	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	24
24.5	[illegible]	[illegible]	314	294	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	24.5
25	210	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	25
25.5	212	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	25.5
26	214	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	26
26.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	26.5
27	219	174	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	27
27.5	221	176	332	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	27.5
28	222	177	[illegible]	314	644	520	831	747	1104	1641	28
28.5	224	[illegible]	[illegible]	317	650	524	838	754	1114	1655	28.5
29	226	180	340	[illegible]	[illegible]	[illegible]	844	[illegible]	1124	[illegible]	29
29.5	228	[illegible]	341	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	29.5
30	230	183	344	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	30
30.5	[illegible]	185	348	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	30.5
31	[illegible]	186	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	31
31.5	[illegible]	188	354	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	31.5
32	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	32
32.5	240	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	32.5
33	242	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1200	[illegible]	33
33.5	244	194	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
34	246	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
34.5	248	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
35	250	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
35.5	252	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
36	254	200	[illegible]	356	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Note: Capacities are given in nearest whole numbers.

TABLE Vb FLOW MEASUREMENT

ORIFICE TABLES

Head in Inches	3" Orifice		4" Orifice		5" Orifice		6" Orifice		7" Orifice	9" Orifice	Head in Inches
	4 in. Pipe	6 in. Pipe	6 in. Pipe	8 in. Pipe	6 in. Pipe	8 in. Pipe	8 in. Pipe	10 in. Pipe	10 in. Pipe	10 in. Pipe	
36.5	256	201	383	356	743	588	937	852	1259	1879	36.5
37	257	203	385	358	748	592	943	857	1266	1893	37
37.5	259	204	[illegible]	[illegible]	754	596	949	862	1274		37.5
38	260	205	390	[illegible]	759	600	955	867	1281		38
38.5	262	206	393	365	765	604	961	872	1289		38.5
39	263	208	396	367	770	608	967	877	1295		39
39.5	265	209	398	369	776	612	974	882	1304		39.5
40	266	210	401	371	781	616	979	887	1311		40
40.5	267	211	403	373	786	620	985	891	1319		40.5
41	269	212	406	375	790	624	990	896	1326		41
41.5	271	213	408	378	795	628	996	901	1334		41.5
42	272	214	411	380	800	631	1001	906	1341		42
42.5	274	216	413	382	805	635	1007	910	1349		42.5
43	275	217	415	384	810	638	1012	915	1356		43
43.5	277	218	418	386	815	642	1018	920	1364		43.5
44	278	219	420	388	820	645	1023	925	1371		44
44.5	280	220	422	390	824	649	1029	929	1379		44.5
45	281	222	425	392	828	652	1034	934	1387		45
45.5	283	223	427	394	832	656	1040	939	1394		45.5
46	284	224	429	396	837	659	1045	944	1401		46
46.5	285	225	432	399	842	663	1051	948	1409		46.5
47	287	227	434	401	847	666	1056	953	1416		47
47.5	289	228	437	403	851	669	1062	958	1424		47.5
48	290	229	440	405	855	672	1067	963	1431		48
48.5	292	230	442	407	859	676	1073	967	1439		48.5
49	293	231	444	409	863	679	1078	972	1446		49
49.5	294	232	446	411	868	683	1084	977	1454		49.5
50	296	234	448	413	872	686	1089	982	1461		50
50.5	298	235	450	415	876	690	1095	986	1469		50.5
51	300	236	453	417	880	693	1100	991	1476		51
51.5	301	237	455	419	884	697	1105	996	1484		51.5
52	302	238	457	421	888	700	1110	1000	1491		52
52.5	303	239	459	423	892	704	1115	1005	1499		52.5
53	304	240	461	425	896	707	1120	1009	1506		53
53.5	305	241	463	427	900	711	1125	1014	1513		53.5
54	307	243	465	429	904	714	1130	1018	1520		54
54.5	309	244	467	431	908	718	1135	1023	1527		54.5
55	310	246	469	433	912	721	1140	1027	1534		55
55.5	311	247	471	435	915	725	1145	1032	1541		55.5
56	313	248	472	437	919	727	1150	1036	1548		56
56.5	314	249	474	439	923	730	1155	1040	1554		56.5
57	315	250	476	441	927	733	1160	1044	1560		57
57.5	316	251	478	443	930	736	1165	1048	1567		57.5
58	317	252	480	445	934	739	1170	1052	1574		58
58.5	319	253	482	447	938	742	1175	1056	1580		58.5
59	320	254	485	449	942	745	1180	1060	1586		59
59.5	321	256	487	451	945	748	1185	1064	1592		59.5
60	323	257	489	453	948	751	1190	1068	1598		60
60.5	324	258	491	455	951	754	1195	1072			60.5
61	325	259	492	457	955	757	1200	1076			61
61.5	326	261	494	459	958	760	1205	1080			61.5
62	328	262	496	461	961	763	1209	1084			62
62.5	329	263	498	463	964	766	1214	1088			62.5
63	330	264	500	465	968	769	1218	1092			63
63.5	331	265	502	467	971	772	1223	1096			63.5
64	333	266	504	469	974	775	1227	1099			64
64.5	334	267	507	471	977	778	1232	1103			64.5
65	335	268	509	472	981	781	1236	1106			65
65.5	336	269	511	474	984	784	1241	1110			65.5
66	338	271	513	475	988	787	1245	1113			66
66.5	339	272	515	477	991	790	1250	1117			66.5
67	340	273	517	479	995	793	1254	1120			67
67.5	341	274	518	481	998	796	1259	1124			67.5
68	343	275	520	483	1002	799	1263	1127			68
68.5	344	276	521	485	1005	802	1268	1131			68.5
69	345	277	523	487	1009	805	1272	1134			69
69.5	347	278	524	489	1012	808	1276	1137			69.5
70	349	280	525	491	1016	811	1280	1140			70

Note: Capacities are given in nearest whole numbers

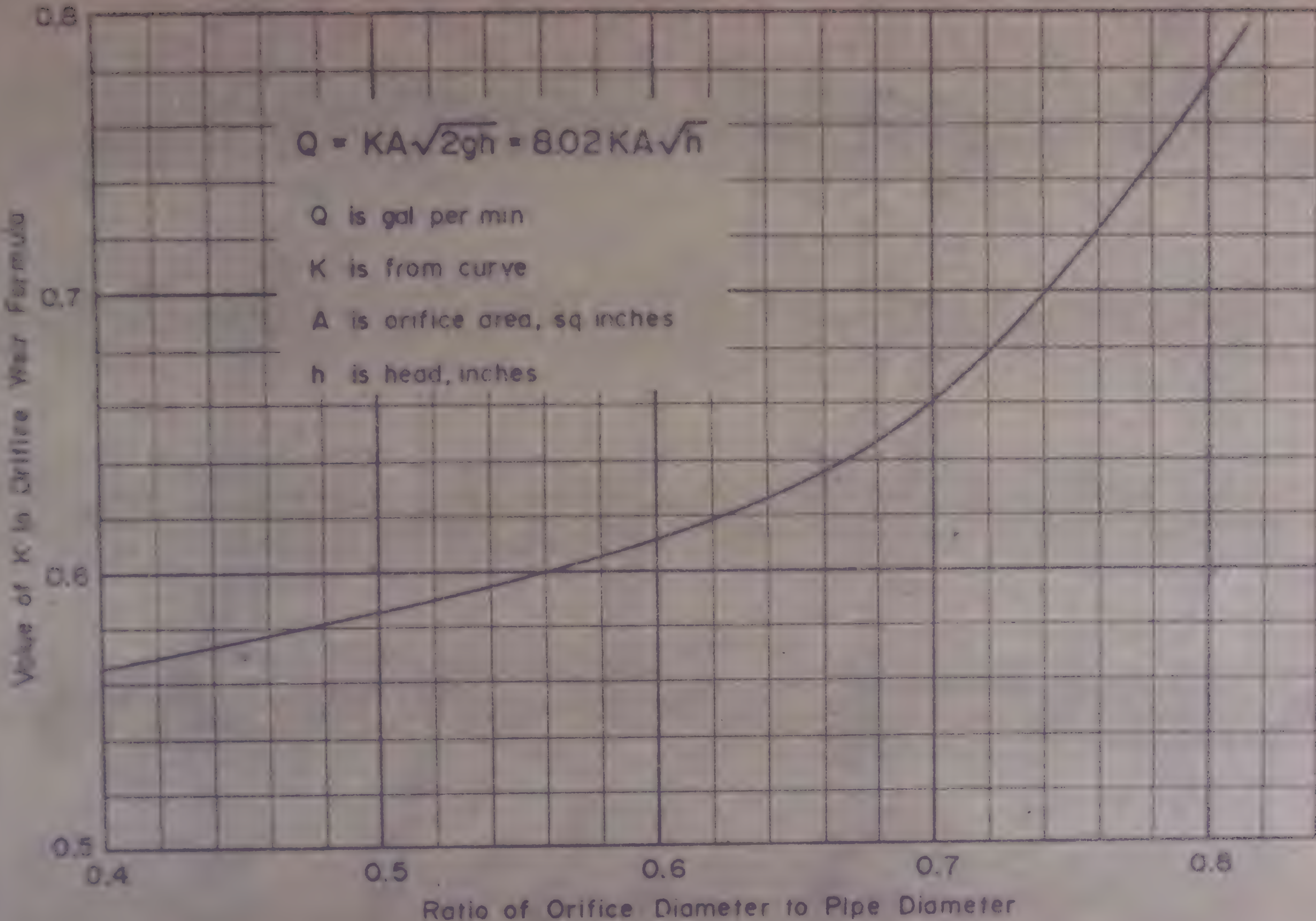


Figure 7. Value of discharge factor, K, in the orifice-weir formula varies with the ratio of orifice diameter as shown by this curve.

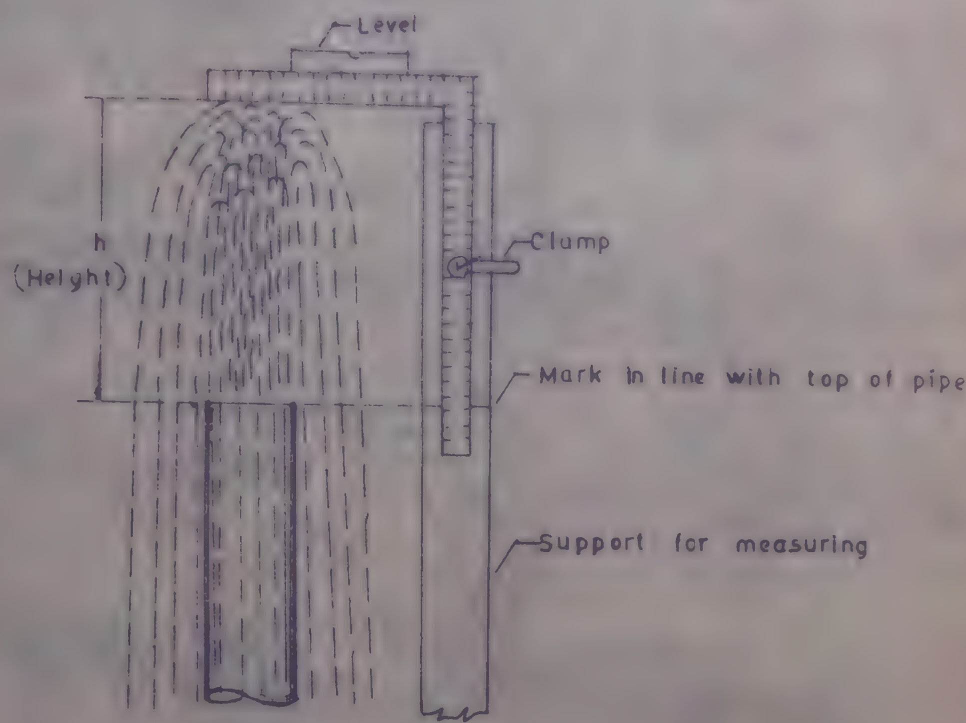


FIG. 8 MEASURING HEIGHT OF CREST OF JET FLOW FROM A VERTICAL PIPE

TABLE 11

DISCHARGE FROM VERTICAL PIPE IN GPM

Height of Crest in inches	Nominal Diameter of Pipe					
	2"	3"	4"	5"	6"	8"
1 ½	22	43	68	85	110	160
2	26	55	93	120	160	230
3	33	74	130	185	250	385
4	38	88	155	230	320	520
5	44	99	175	270	380	630
6	48	110	190	300	430	730
8	56	125	225	360	510	900
10	62	140	255	400	580	1050
12	69	160	280	440	640	1150
15	78	175	315	500	700	1300
18	85	195	350	540	750	1400
21	93	210	380	595	850	1550
24	100	230	400	640	920	1650

through open vertical pipes. Table VI may be used to ontain pre-calculated values. Figure 9 and Table VII are correspondingly for flows through an open horizontal pipe.

Water Level Measurements: Water levels should be measured to the nearest 1/4 inch during pumping, at shorter intervals during the first two hours as follows:

First 5 minutes	-	every ½ minute
Next hour	-	" 5 minutes
Next 2 hours	-	" 20 minutes
Beyond 2 hours	-	" hour

In the observation wells the readings should be taken as follows:

First hour	-	every 2 minutes
Next hour	-	" 5 "
Next 2 hours	-	" 10 "
Beyond 2 hours	-	" 20 "

Automatic recorders may be used for these observations, but may not be readily available. The usual means employed are the electric sounder, the wetted tape method and the air line method. The electric sounder consists of an electrode suspended by 2 insulated wires and a meter indicate a close circuit when water level is reached; torch light batteries supply the current. (Refer Figure 11 (a)). As water level drops, the electrode has to be lowered and the change can be measured on the wire which is marked at 5 ft intervals.

The wetted tape method is a very accurate method which involves lowering of a measuring tape weighted by a lead piece and its lower parts coated with carpenter's chalk. When the tape dips into water the point at which it is held at the measuring point is noted and from it the reading marked on the chalked portion by the water level at the lower end, is subtracted giving an accurate reading of the water level. This can be used for wells upto 100 feet.

FIG. 9 RATE OF FLOW FROM A HORIZONTAL PIPE CAN BE ESTIMATED FROM DISTANCE X

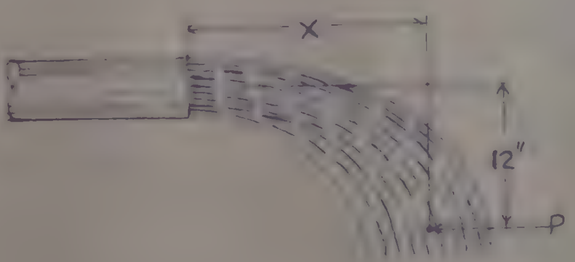


Table. VII DISCHARGE FROM HORIZONTAL PIPE FLOWING FULL IN GPM

Distance X in inches at 12" drop	Pipe diameter					
	2"	3"	4"	5"	6"	8"
6	21	46	80	125	181	312
7	24	54	93	146	211	364
8	28	61	106	167	242	416
9	31	69	119	188	272	468
10	35	77	133	208	302	520
11	38	84	146	229	332	572
12	42	92	159	250	362	624
15	52	115	199	313	453	780
20	70	154	265	417	604	1040

FIG. 10 RELATIONSHIPS BETWEEN PERCENT DRAWDOWN AND YIELD AND BETWEEN PERCENT DRAWDOWN AND SPECIFIC CAPACITY FOR WATER TABLE WELL IN HOMOGENEOUS WATER TABLE AQUIFER

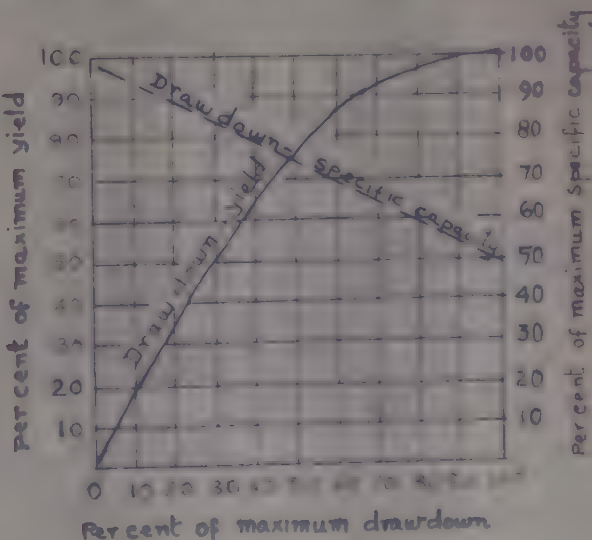


Table. IV WELL DIAMETER VS YIELD RATIO IN %

Well Diameters						
6"	12"	18"	24"	30"	36"	48"
100	110	117	122	127	131	137
—	100	106	111	116	119	125
—	—	100	104	108	112	117
—	—	—	100	104	107	112
—	—	—	—	100	103	108
—	—	—	—	—	100	105

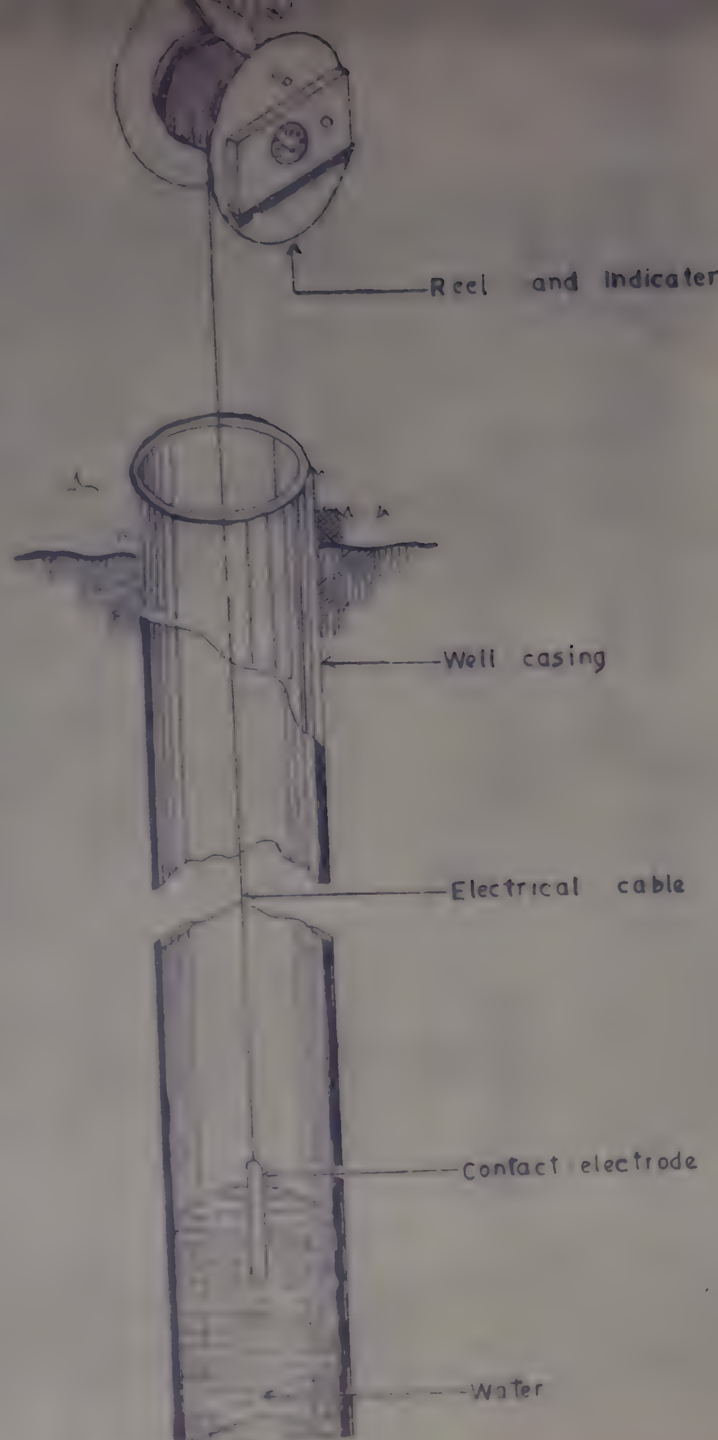


FIG. 11(a) Electric Sounder for measuring depth to water consists of electrode, two wire cable and ammeter which indicates a closed circuit when electrode touches water.

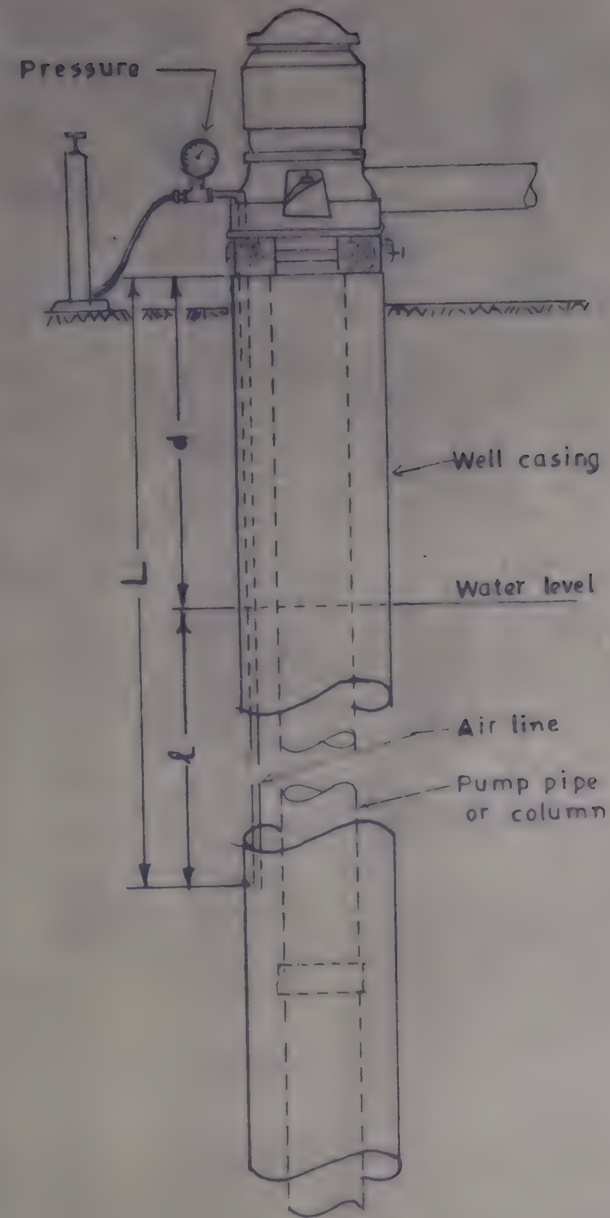


FIG 11(b) Typical installation for measuring waterlevels by air-line method

The air line consists of a small diameter pipe (commonly copper or brass) from the top of the well to several feet below the lowest water level anticipated; the exact length of the pipe should be measured. The pipe should be absolutely vertical and airtight. The upper end is fitted in suitable connections so that a valve and ordinary tyre pump can be fitted and air can be pumped into the tube. A tee is provided to which an air pressure guage is attached as shown in figure 11 (b) preferably showing feet of water rather than pound per square inch (psi). This works on the principle that the air pressure required to push all water in the submerged portion of the tube equals water pressure of a column of water of that height. Thus air is sent into the air-line until the guage shows a maximum constant, indicating that all the water has been forced out and water pressure expressed in feet of water shown by the guage indicates the water level. This may be calculated from:

$d = L - 1$

where d = depth to water
L = Length of air line
and l = pressure head represented by a column of water of height equal to the submerged length of air line.

Estimation of extractable amounts from a well:

As already mentioned a test on a well is carried out for capacity, measurements of depth to water and pump discharge. If a well is pumped at the rate of 200 gpm with 25 feet drawdown, the specific capacity would be 8 gpm per foot drawdown; this is not constant for various pumps discharges. It does not follow that if the drawdown is doubled the pumping rate would be doubled to 400 gpm. Hence it is desirable to measure drawdown at different pumping rates plotting a curve of drawdown against pumping rates. The water levels should be measured at different rates and always ensuring that the pumping water level

has stabilised long enough at each pumping rate before the discharge is increased. This may hence require several hours or days of continuous pumping. It is also advisable to record recovery of the water levels. As the plantations would need only such simple tests, the detailed tests for determination of aquifer parameters like permeability etc., which require installation of observation wells has not been dealt with here. The tests for determining capacity should be carried out for at least 4 different pumping rates.

The extractable water from a well is generally calculated for 50% drawdowns so as to avoid possible overdraft. Figure 10 shows the relation between percent drawdown and percentage yield and between percent drawdown and specific capacity in an homogenous water table aquifer.

It can be seen that it is uneconomical to operate a well with a drawdown greater than 70% of the maximum. At 70%, 92% of the maximum yield is obtained i.e. within 8% of the maximum. To obtain the additional 8% requires an additional 30% of drawdown: the extra cost in pumping would be out of proportion to the increase in yield. On establishing the known extractable yield from a well and determining the total head, a suitable pump can be selected.

Investigation for and Estimation of Surface Water:

As mentioned earlier, in estates, the major sources of water occur in the form of surface water because of the steep topography and other morphological conditions. Surface water originates mostly from rainfall and contribution from ground water. These include small upland streams and rivers through valleys. The stream would be perennial if fed by springs. Before we proceed further it is necessary to familiarize oneself with certain terms related to surface water.

Precipitation: is the rainfall measured in inches or centimetres and may be recorded hourly, daily, monthly or annually as desired. The depth of rainfall denotes the depth of total rainwater spread over the entire area for which the measurements are applicable. Periodical rainfall data collection is of utmost importance for any water resources scheme. The data may be analysed for different purposes eg, drought prediction, crop design, planning for impounding structures for irrigation and water supply schemes, erosion control and so on.

Runoff: is the portion of the precipitation that flows on the surface or under the surface of soils towards streams. This is called the surface runoff and sub-surface runoff or base flow respectively. The runoff from watersheds maybe classified into two groups as storm run-off and base flow. The storm runoff is the runoff during and immediately after a storm or very short period of heavy precipitation. A portion of the precipitation would be absorbed by the soil cover of the watershed; part of this is released later at a more or less constant rate in the form of affluent seepage into the stream courses causing the base flow. Though perennial springs from rocks also add to the base flow, this is normally not included in the run-off calculations.

Annual Yield from Water-Shed: This is the total volume of surface run off due to the cumulative effect of the rainfall in a year.

The major factor that determines the annual yield is the annual rainfall, water-shed area, soil cover and physiography. There are a number of emperical formulae to determine the annual runoff for different parts of the world. However there is no single universal formula yet as the parameters are complex and varying in their absolute value. Thus for each water-shed we could derive a unique formula. However, for the plantations of South India, the formuladeveloped by ICAR for the Nilgiris may be applicable:

$$R = 0.4564\ P\ A^{-0.0837}$$

where R = Total runoff in cms.

P = Annual rainfall in cms.

A = Watershed area in sq.km.

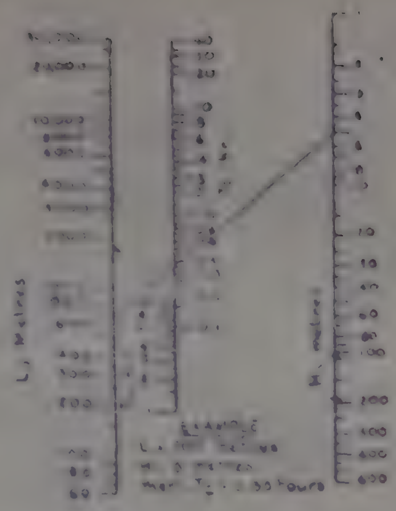
The total volume in cubic metres is obtained by:

$$V = \frac{RA \times 1000 \times 1000}{100}$$

$$= \underline{RA \times 10^4 \text{ cubic metres}}$$

Peak Discharge: The rate of run-off is called discharge. This is influenced by several complex parameters like intensity of rainfall, soil characteristics, steepness and shape of the watershed, vegetal cover, land treatment factors (like terraced lands)etc. For a particular watershed the peak discharge is the maximum rate of runoff for a given duration and intensity of rainfall. Several formulae are used to determine the peak discharge for the purposes of designing structures across streams. Determination of peak discharge involves choosing the maximum rainfall intensity that may occur in a given return period. This period is usually the number of years for which the structure is to be designed. The meteorological department has ready reference maps of rainfall intensity of India for different return periods.

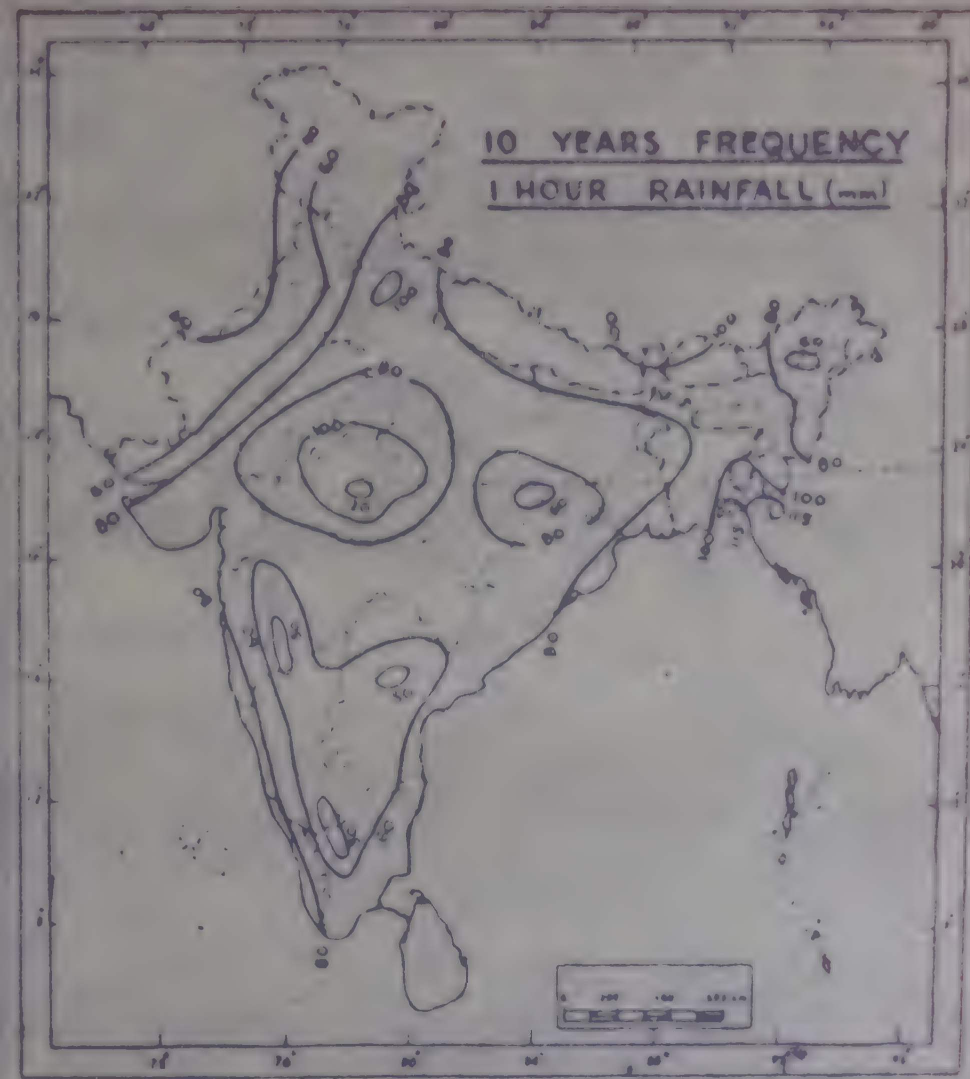
A ready reckoner nomograph to determine the peak discharge is given in Figures 2(a) to 2(d). A step by step method to use this graph is given in Table VI. However for assessment of



L = Maximum length of flow in metres

H = Difference in elevation between most remote point and outlet excluding sudden drops in metres.

Fig 12(a) Nomograph for estimating time of concentration (converted from National Engineering Handbook—Section 4, Hydrology Supplement A—Soil Conservation Service, U.S. Department Agri., 1957).

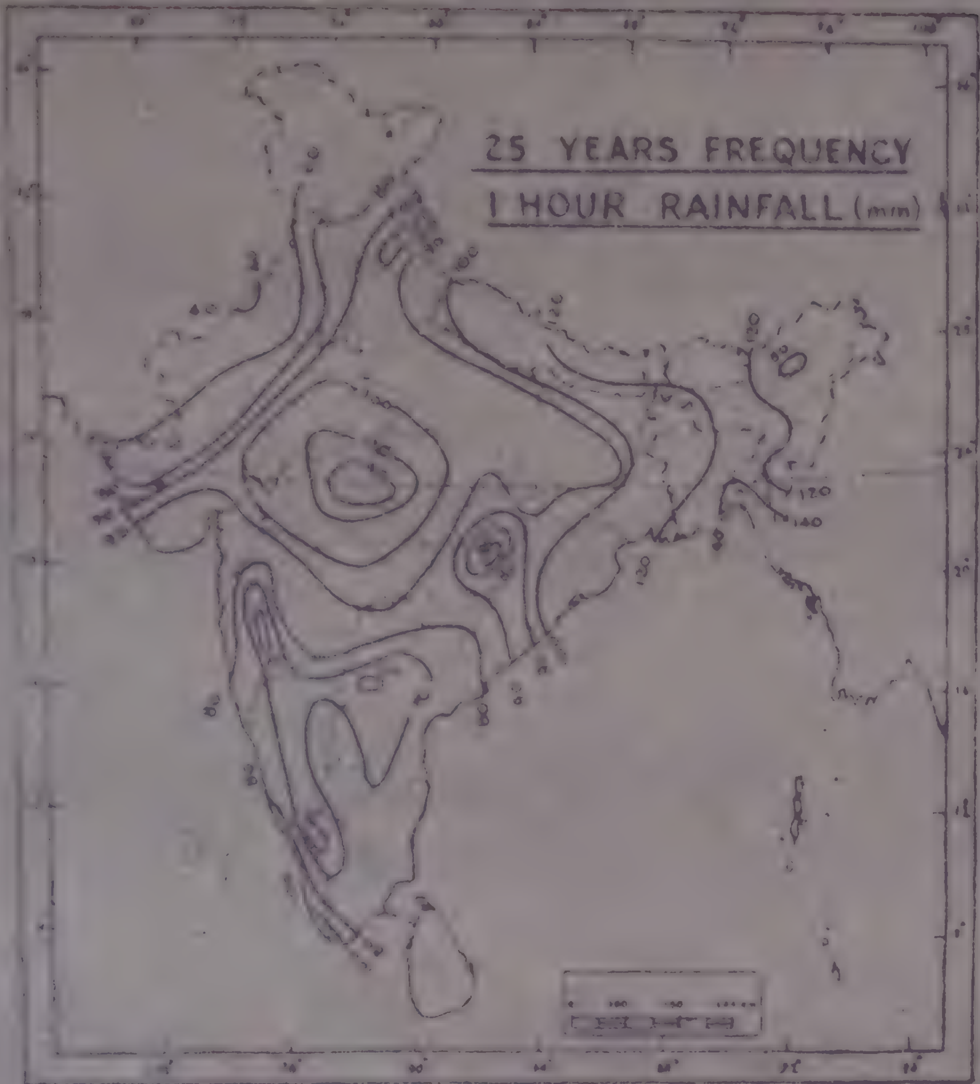


Based upon Survey of India map with the permission of the Surveyor General of India

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

The boundary of Meghalaya shown on this map is as interpreted from the North-Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.

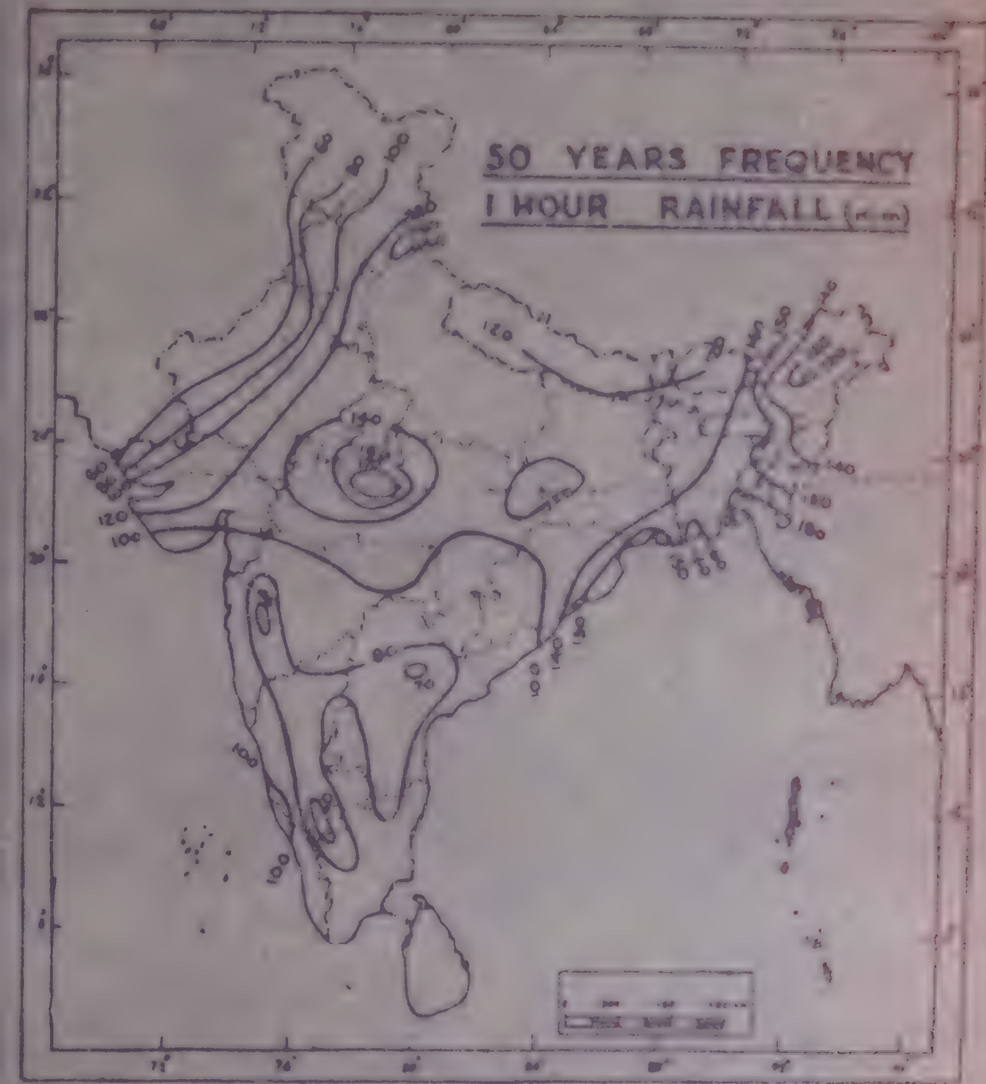
Fig 12(b)(i) 10 years frequency— hour rainfall (mm)



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

The boundary of Meghalaya shown on this map is as interpreted from the North Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.

Fig(2) (ii) 25 years frequency—1 hour rainfall (mm)



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

The boundary of Meghalaya shown on this map is as interpreted from the North Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.

Fig(2) (iii) 50 years frequency—1 hour rainfall (mm)

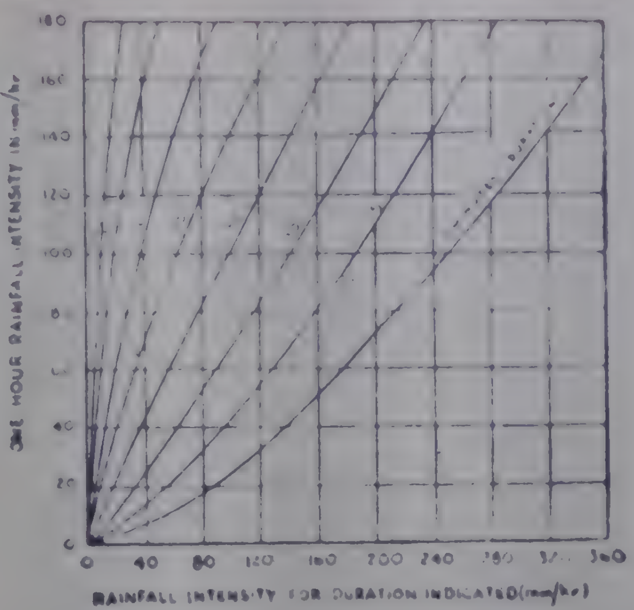


Fig.12(c) Relationship of one hour rainfall intensities at other durations (converted and redrawn from G. A. Mathway, Military Air Field-design of drainage facilities, Trans. Am. Soc. Civil Engineers 110 : 700. 1945).

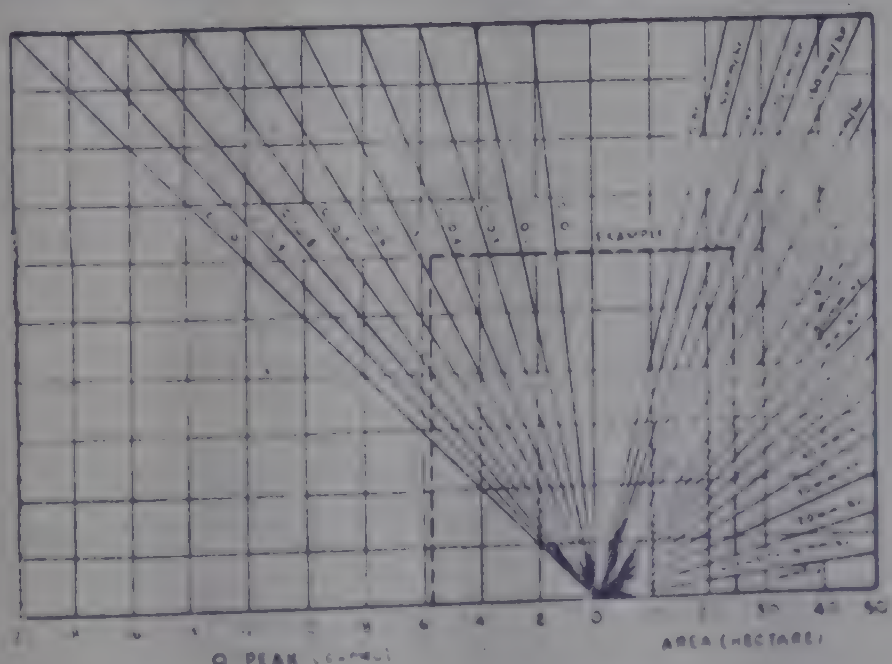


Fig.12 (d) Chart for Estimation of Q peak by Rational Method.

TABLE V.

Rational Method

$$Q = \frac{1}{360} CIA$$

Where, Q = Peak rate of runoff in cubic metres per second (cumec) for a given frequency of rainfall

C = Runoff co-efficient [(Table I (*a*)].

I = Intensity in mm per hour for design frequency for duration equal to time of concentration (Tc).

A = Area of watershed in hectares.

Procedure :

(1) Determine the area under various land uses and soils.

TABLE I (*a*). VALUES OF C FOR USE IN RATIONAL FORMULA

Soil type	*Land use*		
	Cultivated	*Pasture*	*Forest*
With above average infiltration rate, usually sandy or gravelly	0·29	0·15	0·10
With average infiltration rate, no clay pans, loams and similar soils	0·40	0·35	0·30
With below average infiltration rate, heavy clay soils or soils with a clay pan near the surface, shallow soils above impervious rock	0·50	0·45	0·40

(2) Compute weighted 'C' for the watershed:

$$C = \frac{A_1C_1 + A_2C_2 + A_3C_3 + \text{------}}{A}$$

Where, A_1, A_2 and A_3 etc. are the areas in ha under various landuses and soil types having corresponding value of C_1, C_2 and C_3 etc. respectively [from Table I (*a*)].

and, A = Total area of watershed in ha.

(3) Determine the time of concentration (Tc) from Fig. 3 (*a*) by using maximum length of flow and fall along the line.

(4) Determine the 1 hour intensity for the design frequency from Fig. 3 (*b*).

(5) Convert 1 hour intensity to intensity for duration equal to time of concentration from Fig. 3 (*c*).

(6) Using Fig. 3 (*d*), estimate the peak rate of runoff, enter at area of the watershed in ha to the intensity value determined in step 5, turn horizontal to the weighted value of 'C' and read along Q peak.

Example :

Estimate peak rate of runoff for 10 years frequency from 25 ha watershed in medium black soil having 15, 5 and 5 ha under cultivation, forest and grass cover respectively. The area is located at 76° longitude and 24° latitude. The elevation of the highest and outlet points is 250 and 245 m respectively and the maximum length of run is 700 m.

(1) $C = \frac{A_1C_1 + A_2C_2 + A_3C_3}{A}$

$= \frac{15 \times 0.5 + 5 \times 0.4 + 5 \times 0.45}{25}$

$= 0.47$

(2) For L = 700m, H = 5m, Tc = 0.35 hour = 21 minutes [from Fig. 3 (*a*)]

(3) 1 hour rainfall intensity for 10 year frequency at defined location = 105 mm/hour [from Fig. 3 (*b*) (i)].

(4) $I_{Tc} = I_{21}$ minutes = 175 mm/hour [from Fig. 3 (*c*)].

(5) Peak rate of runoff = 5.7 cumec (Q peak) [as shown in Fig. 3 (*d*)]

surface water the peak discharge has little relevance.

Base Flow: of any stream is very important and can be measured directly with simple techniques. The base flow measurement data (stream guaging data) for different months of a year, collected for a period even as short as 5 years gives very relaiable information for designing water supply schemes. The base flow is maximum immediately after the monsoons and gradually decreases to a minimum during peak summer. The fact that some streams cease to flow a few months after the monsoons does not necessarily imply that data of base flow for these streams are not dependable. With proper planning and suitable structures this water can be used when available, while the excess can be conserved for use during lean periods. The simple techniques for measuring the surface flow in streams are dealt with separately.

Rainfall analysis: The rainfall analysis as mentioned earlier may be done for various purposes. At present, we may be using rainfall data for limited purposes only. But with the advancement of technology and increasing demands every aspect of rainfall, like its hourly, daily and monthly distribution, its consistency etc., may have to be minutely studied and analysed. Precautions may have to be taken for even the smallest deviation from the projected trend. Therefore, rainfall data should be recorded and maintained promptly; incidently this does not involve any major expenses.

In plantations, the rainfall analysis can be done for the following purposes;

(a) Estimation of runoff from watersheds
(b) Determination of net water requirements for plantation crops for different months.
(c) Estimation of runoff from watersheds.

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Prediction of drought years, its severity and return periods will help in justifying the extra funds required for buffer storage of water, conservation measures, exploitation of perennial sources at extra cost, etc.

It may be noted that drought is a relative term and the concept that places having higher rainfall will have lesser or no droughts is baseless. Drought is a phenomenon during which the moisture content of the soil, the water table and the baseflow in streams gets depleted to such an extent that plants and all living creatures find it difficult to survive. Drought occurs normally at places where the variability of rainfall is very high.

The formulae and statistical methods of drought analysis is not dealt with, as this is beyond the scope of this Seminar. For calculation of the available surface water from rainfall and subsequent runoff in plantations, the mean annual rainfall figure is sufficient.

Evaporation and Evapotranspiration:

Evaporation takes place from all surface water bodies at different rates for different times of the year. The evaporation losses are of significance as far as open storage reservoirs are concerned, though it is not so for the schemes based on the flow in streams, wells, springs, etc. In the design of open reservoirs the net capacity is arrived at after deducting the total annual evaporation, measured in cms, or inches. Though there are several complicated formulae to determine the rate of evaporation, the most practical and useful is that proposed by Rohwer (1931)

$$E = C\ (1.465 - 0.0186\ B)\ (0.44 + 0.118\ W)\ (e_s - e_d)$$

where E = Evaporation in inches

C = Coefficient varying between 0.5 to 0.99 (usually taken as 0.75)

B = The mean barometric pressure at $32^{\circ}F$ in inches of mercury bar.

W = Mean wind velocity in mph

e_s = Mean vapour pressure of standard vapour at temperature of water surface in inches of mercury bar.

$e_d = e_s$ x relative humidity. The following Table, VI gives the approximate values of e_s for different air temperatures.

TABLE VI

Air temperature in ^oF	e_s	Air temperature in ^oF	e_s
0	0.0383	50	0.360
5	0.0491	55	0.432
10	0.0631	60	0.517
15	0.0810	65	0.616
20	0.103	70	0.732
25	0.130	75	0.866
30	0.164	80	0.926
35	0.203	85	1.201
40	0.247	90	1.408
45	0.298	95	1.645
		100	1.916

The values of relative humidity, mean pressure, mean wind velocity etc., can be obtained from the nearest meteorological station or base. Care should be taken to convert all the values into the units specified in the above formula.

Evapotranspiration is the combination of evaporation from the soil and transpiration from plants/trees. Though this is significant in estates it does not appreciably affect the losses from reservoirs which we are more concerned with as far as water supply is concerned.

Simple methods of measuring stream flow:

The stream flow is measured periodically at regualar intervals

preferably daily during post monsoon months and fortnightly during summer months. The discharge data is recorded and maintained separately for different streams. The exact location of the guaging point should be marked on the estate map. The standard units of discharge are: Litres per second(lps), cubic feet per second(cusec), cubic metres per second (cumec), and gallons per hour (gph). The various methods applicable to plantation streams/rivers are:

(a) Area velocity method (Float method)

(b) Rectangular notch

(c) V-notch

(a) Area Velocity Method: This method involves measurement of velocity of the flow using a float. The time taken for a float to travel a given distance (minimum 20m) is noted using a stop watch. The velocity is obtained by dividing the distance by the time taken to travel (in seconds). The area of cross section of the water is calculated thus:

The average water depth is determined by measuring depth of water at an interval of 0.25 metre across the width of the stream. The area obtained in sq.m is a product of the average depth of water and stream width (wetted portion).

The discharge is then calculated by:

$Q = 0.8\ AV$

where 0.8 is a velocity correction factor.

Q = discharge in cumec

A = Cross sectional area in m^2

V = Velocity in metres/sec.

This method should be adopted where;

- the stream has a relatively straight course of atleast 30 metres.
- the entire flow should be through a single channel with almost uniform width.
- the flow should not be very turbulant.

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(b) Rectangular weir: This method is adopted for discharges in the range of 0.05 to 0.15 cumecs and involves in controlling the entire flow over a fixed rectangular constriction. The details of the construction are given in figure 13 and is based on the following formula:

$$Q = 0.0184\ LH^{3/2}$$

where Q = Discharge, lps
L = Length of opening, cms
H = Height of water over weir, cms.

The Table VII gives values of discharges through contracted rectangular weirs.

(c) V- notch weir: The principle involved is the same as that of a rectangualr weir except ... the opening will be in the shape of the letter V. The most commonly used V-notch is a 90^0V-notch. These are used to measure discharges less than 50 lps, And are based on the formula:

$$Q = 0.0138\ H^{5/2}$$

where Q = discharge, lps
H = Height of water over the V-notch.cms.

Figures 14 (a) and 14 (b) give the details of construction and installation of a V-notch. The Table VIII gives the values of discharges for different heights of water over the V-notch.

Assessment of flow through the stream bed material:

If the stream bed consists of sand, gravel and like mate -rials , there is a possibility of some sub-surface flow when the surface flow is nil or negligible. To assess this there may be several formulae and indirect methods, the most reliable is by actual pumping.

A pit with a diameter of 2m (at bottom) and depth to the impervious layer or 3m, whichever is less, is dug in the

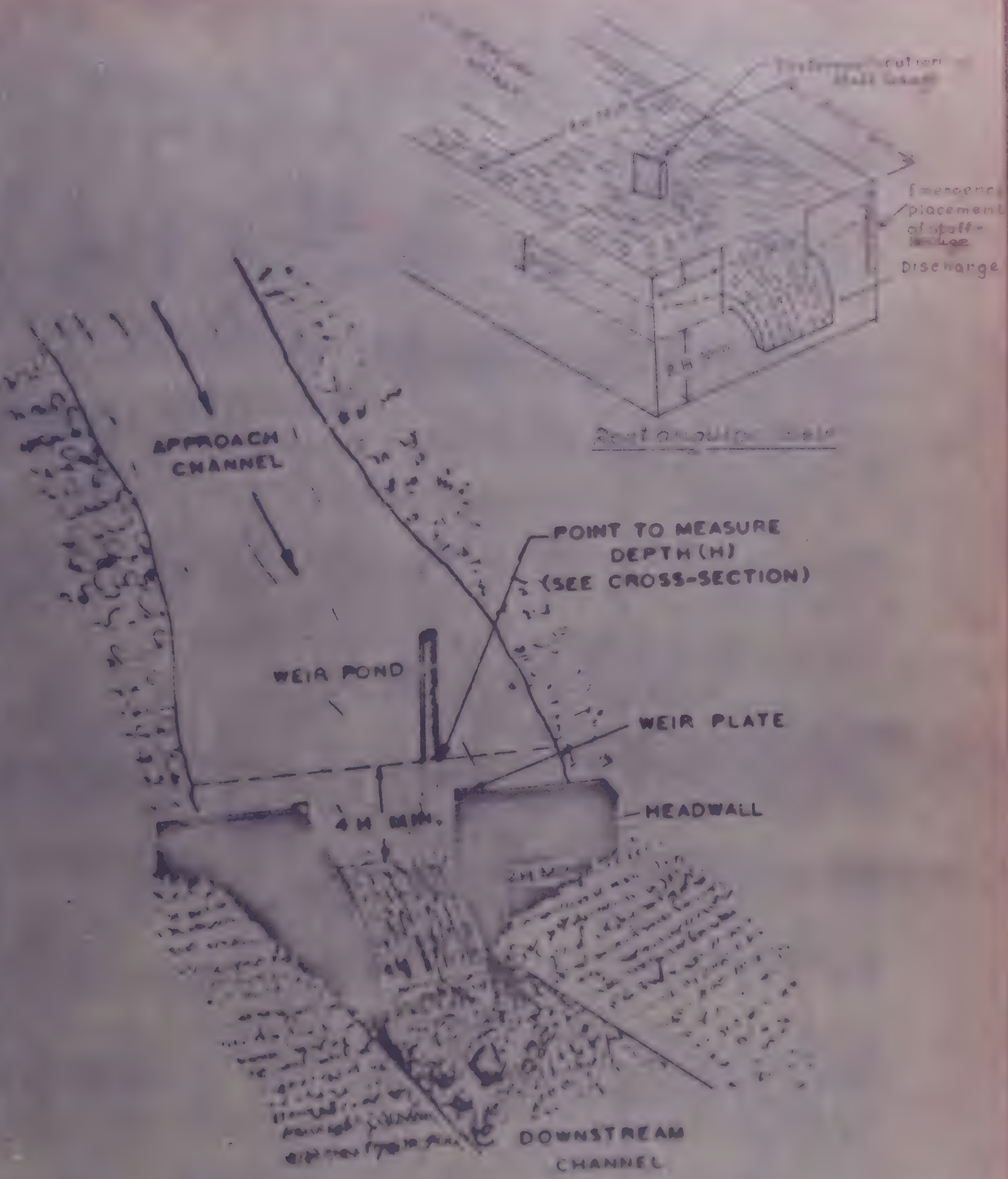

Fig. 13. A rectangular weir with end contractions installed in a field channel.

Table VII: Discharge through contracted rectangular weirs
litres per second

Head over weir cm	Width of weir			
	30 cm	40 cm	50 cm	60 cm
5.0	5.97	8.0	10.1	12.2
5.5	6.9	9.3	11.6	14.0
6.0	7.8	10.5	13.1	15.9
6.5	8.4	11.8	14.9	17.9
7.0	9.7	13.2	16.6	20.0
7.5	10.7	14.5	18.3	22.1
8.0	11.8	16.0	20.1	24.3
8.5	12.9	17.6	22.1	26.7
9.0	14.0	19.0	24.0	28.9
9.5	15.2	20.7	26.0	31.2
10.0	16.3	22.2	28.0	33.8
10.5	17.5	23.7	30.0	36.2
11.0	18.7	25.3	32.0	37.7
11.5	19.9	27.1	34.3	41.4
12.0	21.3	29.0	36.7	44.4
12.5	22.5	30.7	39.0	47.1
13.0	23.7	32.3	40.9	49.5
13.5	24.8	34.0	43.0	52.2
14.0	26.2	35.8	45.4	55.2
14.5	27.7	37.9	48.2	58.5
15.0	28.8	39.5	50.3	60.9
16.0	31.6	43.3	55.2	67.0
17.0	34.3	47.2	60.1	73.0
18.0	37.0	51.0	65.3	79.0
19.0	39.8	55.0	70.2	85.3
20.0	42.8	59.3	75.8	88.8
21.0	45.7	63.3	81.0	99.0
22.0	48.7	67.5	86.7	105.7
23.0	51.3	71.7	92.2	112.3
24.0	54.7	76.5	94.8	120.0
25.0	57.0	79.8	102.7	125.8
26.0	60.3	84.6	109.2	133.3
27.0	63.5	89.2	115.0	140.8
28.0	66.5	93.7	122.2	148.3
29.0	69.5	98.3	127.0	155.7
30.0	72.5	102.7	133.0	163.3

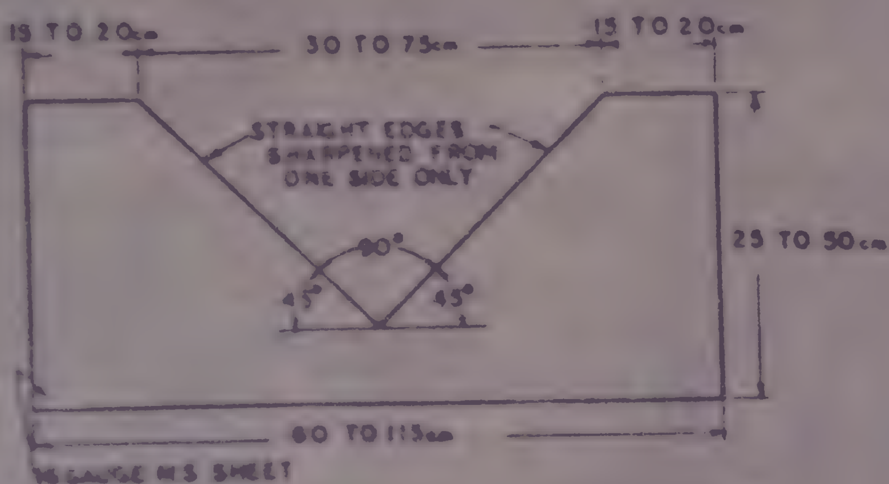

Fig (A)(a) Details of a 90° V-notch weir.

Fig (A)(b) A 90° V-notch weir installed in a field channel. Note the position of the scale used to measure the head.

Table 4VIII Discharge through a 90°V-notch

Height of water over V-notch cm	Discharge litres/second	Height of water over V-notch cm	Discharge litres/second	Height of water over V-notch cm	Discharge litres/second
4.0	0.45	13.0	8.6	22.0	31.0
4.5	0.60	13.5	9.5	22.5	34.0
5.0	0.80	14.0	10.5	23.0	35.7
5.5	1.0	14.5	11.3	23.5	38.2
6.0	1.2	15.0	12.3	24.0	40.0
6.5	1.5	15.5	13.3	24.5	42.7
7.0	1.8	16.0	14.5	25.0	44.5
7.5	2.2	16.5	15.6	25.5	46.7
8.0	2.5	17.0	16.7	26.0	48.8
8.5	2.8	17.5	18.3	26.5	51.0
9.0	3.4	18.0	19.4	27.0	53.8
9.5	3.9	18.5	21.7	27.5	56.3
10.0	4.5	19.0	22.3	28.0	58.7
10.5	5.1	19.5	23.5	28.5	61.5
11.0	5.7	20.0	25.5	29.0	64.5
11.5	6.3	20.5	27.0	29.5	66.8
12.0	7.1	21.0	28.3	30.0	69.4
12.5	7.8	21.5	30.3		

stream bed when the stream ceases to flow.

The procedure is similar to that for estimating yields in open wells dealt with under " Determination of Well Yields" described earlier. However, since the bed material is expected to be sandy formation with a comparitively high flow, unlikewells in rocks, the permissable drawdown could be taken as 70%.

Assessment of extractable amount of water from each source:

Having assessed the amount of surface flow and sub-surface flow, it is important to decide the amount of water that should be extracted from each source. It is not advisable to extract entirely the available water. However, it is difficult to lay down a norm as to how much water any individual or any group are eligible to tap from a surface water source. This is a policy matter to be decided by Governmental authorities and vary from catchment to catchment. However, we may lay down a norm that any stream or river entering into the estates from a distance should be subject to an extraction of only 20% of its minimum flow. An estate may or may not have full rights over a stream originating within it. In short it is best first to obtain clearance from the competent authority before extraction.

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SESSION III

PLANNING
FOR
WATER SUPPLY
SYSTEMS IN
PLANTATIONS

The Need for Planning:

Several water supply schemes have proved to be ineffective or have failed to function smoothly due to improper or no planning. Whether rural or urban, adhoc measures have been resorted to in trying to set right water supply systems. Though this may yield temporary solutions to various problems, in the long run however, these may have added to the malfunctions of the already ineffective schemes.

Having properly assessed the requirement and the available water resources for extraction, planning involves:

(a) Selection of optimum number and appropriate locations of points of extraction.

(b) Selection of mode of appropriate treatment depending on the quality of each source.

(c) Adoption of appropriate layout of conveyance and supply network based on availability and quality of each source and the requirement and proximity of consumer points.

(d) Inclusion of an efficient sanitary and sewage disposal scheme.

(e) Preparation of a Water Supply Map of the Estate for future guidance and design.

While planning it is imperative to bear in mind that the cumulative available resources should be distributed equitably. This does not mean however that the total available resources are necessarily pooled at one point before distribution. To illustrate this, an estate has three sources, A,B and C for three consumer points a,b, and c respectively. However A does not meet the requirement of a, B meets b's requirement, whereas C has a surplus which can clear the defict of A. Thus the cumulative resources A,B, and C meet the requirement of the whole estate.

Topographical Map of an Estate:

A base map showing all the physical details of an estate is indispensable for planning not only for water supply but also

for many other future purposes. Such a map is called a topographical map. A map to the scale of 1: 3000 would serve the purpose of a base map for planning for water supply in estates. This map should indicate:

(a) The contour lines with atleast 10m interval
(b) All buildings
(c) Roads
(d) Streams and Springs
(e) Areas under different plantations.

Location of sites for extraction of water:

The sites of extraction should obviously be so located so as to provide:
(1) adequate supplies
(2) better quality of supplies as far as possible.
(3) sites as near to the consumer points as possible

Site locations for Groundwater Extraction:

Groundwater resources may be exploited by means of wells and by tapping supplies from springs. The type of well would be determined by various factors like topography, geology, structure of formations etc. In most plantations, shallow open wells would be favourable; the yield from weak aquifers being low, large storages would be required. In some favourable settings bore holes could be considered where deep zones are expected. Springs may occur in many areas.

Open wells would be sited at the lowest points possible as the natural surface as well as sub-surface water would drain towrds these points. In most cases the perennial flow being maintained in the stream enhances groundwater supplies in a well near its source. Besides, the water table generally follows the surface of the ground; places on higher elevations may be expected to tap water at relatively greater depths and in lesser quantities. Whilst the well needs to be as near to the river (or stream) as

possible, it must be protected from pollution and so should be away from direct surface flows which are likely to be highly polluted.

Drainage from areas above the wells should be diverted away. The water then percolating through the soil medium is expected to be naturally filtered. Wells should be at least 50 feet away from toilets, cesspools etc.. It has been studied and concluded that faecal bacteria travels through fine sand upto 25 feet. The location should also be so selected so that direct inflow is prevented at times of flooding. A parapet wall about 2 feet above ground level around the well would be an added protection. The ground should also slope away from the well. When selecting the site, one should also bear in mind geological conditions: a site where a deeper weathered zone is suspected would be preferably suited; well to well spacing should be 500 feet normally but depending on local conditions it may be sited as near as 300 feet or less.

Springs (see figure 15) should be tapped as close as possible to the point where they emerge at the ground surface by construction of a suitable storage chamber. The surface drainage from above this point should be diverted to avoid contamination, Of course since the choice of the site of a spring is not ours, adequate precautions to site latrines, etc., away from springs, should be taken.

Site Locations for Surface Water Extractions:

Having assessed the amount of surface water, both in the form of base flow and the net yield from the water shed it should be possible to locate the point of extraction keeping in view the points discussed earlier.

River Lift: River lift points are located near the streams/rivers having a continuous minimum surface flow throughout the year. The other points to be considered are:

(a) The flow should be concentrated near the bank of extraction.

(b) The point of extraction to be located at or as near as possible to the point where the depth of flowing water is maximum.

(c) The extraction points are best suited at points where there is a pond /pool in the stream which collects all the flow before allowing overflow downstream.

Jack Well: Jack wells are located on the banks of streams with surface flows for a few months and have sufficient sub-surface flow through the sand-bed material, during the rest of the year. The criteria for selection of the points for jackwell are:

(i) Good thickness of sand, gravel or unconsolidated sediments.

(ii) Good thickness of loose overburden with high sand ratio.

(iii) Availability of foundation strata within reasonable depth.

Layout of Conveyance and supply network:

Once the locations of the points of extraction, units of treatment, and the consumer points are determined, the layout of conveyance system can be done with the help of the base map, as given below:

1. The consumer points to be covered by each point of extraction are grouped together on the basis of availability at source and requirement, proximity and accessibility of the consumer points.

2. The points of delivery are marked for each group. These may be:

(a) The consumer point itself, where one single separate pipe may have to be laid to the consumer point from the source.

(b) A point almost equidistant from all the consumer points. This point should necessarily be in the range of 10-20 metres above the highest consumer point of that group.

If the highest consumer point of a group is more than 20 metres above delivery point as mentioned above, it is advisable to

provide secondary pumping stage from the delivery point to the highest consumer point as shown on the map of a Typical Estate (figure 16).

Once the points are determined and marked on the base map, the layout is completed by connecting these points by straight lines. The pressure pipes and gravity pipes are marked separately. For further illustration in this regard refer again to figure 16.

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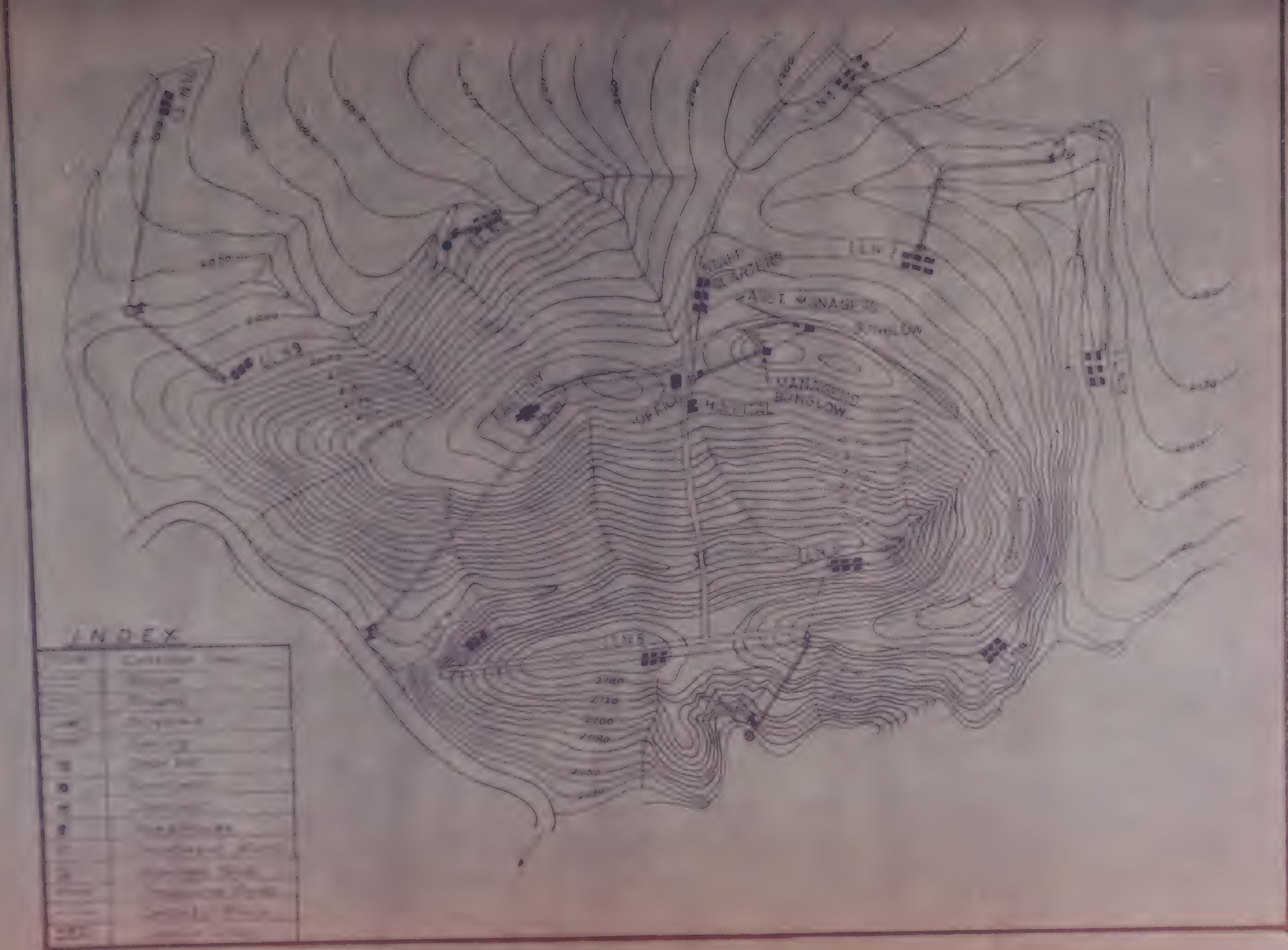
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SESSION IV (A)

DESIGN OF WATER SUPPLY SYSTEMS IN PLANTATIONS

Intake Structures: On completion of the assessment and planning of a water supply system, proper designs for intake structures to tap, store, collect, or impound water must be drawn up as the first step. The proper design of intake structures are of primary importance as defective designs result in over- or under utilisation of the resource resulting further in adoption of adhoc measures at higher recurring costs. Intake structures for plantation water supply schemes are:

(a) Protected storage chambers for springs.

(b) Open wells

(c) Bore wells

(d) Weirs

(e) Jack/Collector wells.

Springs: The springs issuing out from fractures or joints in rocks are best suited for tapping because:

- They are of relatively good quality requiring little treatment.
- They are more reliable and continuous, seldom influenced by the low rainfall of a particular year.

In plantations, they occur in the mountain streambeds and steep escarpments. The other types of springs in plantations are in fact the sub-surface runoff through the soil cover which add to stream flow. Supplies from such springs are often contaminated, and highly influenced by the variation of rainfall.

Spring protection structures discussed in this section are only those occurring at the sides of the steep escarpment and if tapped at this point have no scope of contributing to the surface water. The other kinds of springs occurring in the streambeds maybe tapped by a jack well. the design details of which are discussed later.

Spring protection structure: A simple structure to protect and tap springs from rocks on the steep escarpments, is a square chamber constructed over the actual point where the spring issues at the ground surfaces. The sectional view is shown in figure

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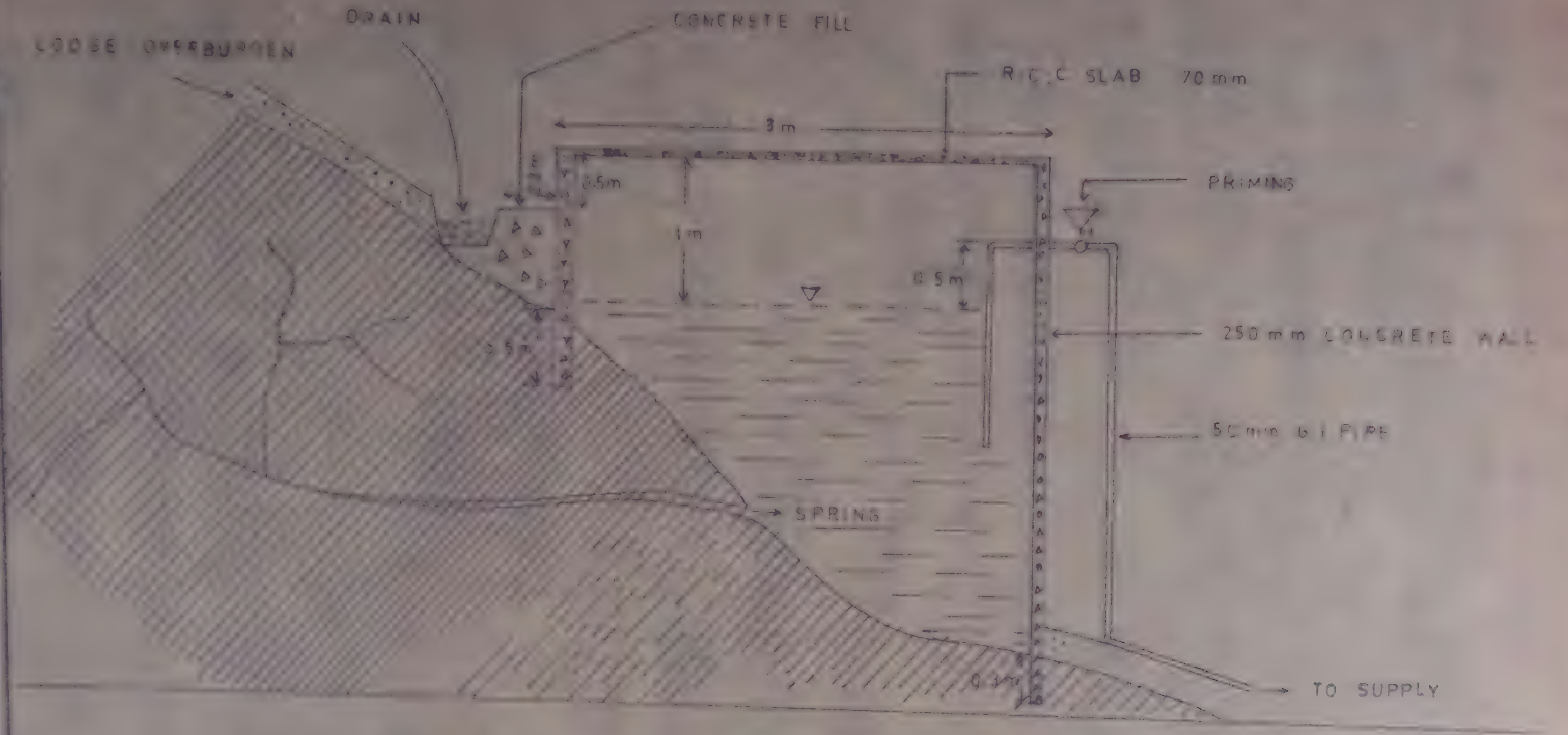
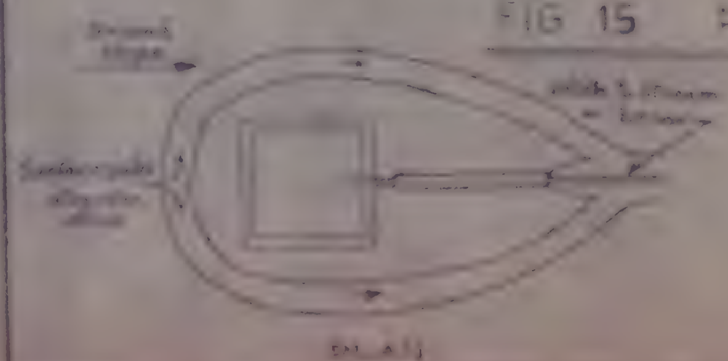


FIG 15 PROTECTION OF A MOUNTAIN SPRING



15. The standard size is 3M X 3M. The height of the chamber depends on the water level in the chamber and the ground slope. The walls are of concrete and should be keyed to the rock by a minimum of 0.3M. The surface water from above the spring should not be allowed to accumulate near the chamber. A drain on all sides of the chamber may be provided for this purpose. The R.c.c. slab on top will have a manhole of 0.8M X 0.8M for periodical inspection. The bottom of the chamber should be excavated till the fresh rock is encountered.

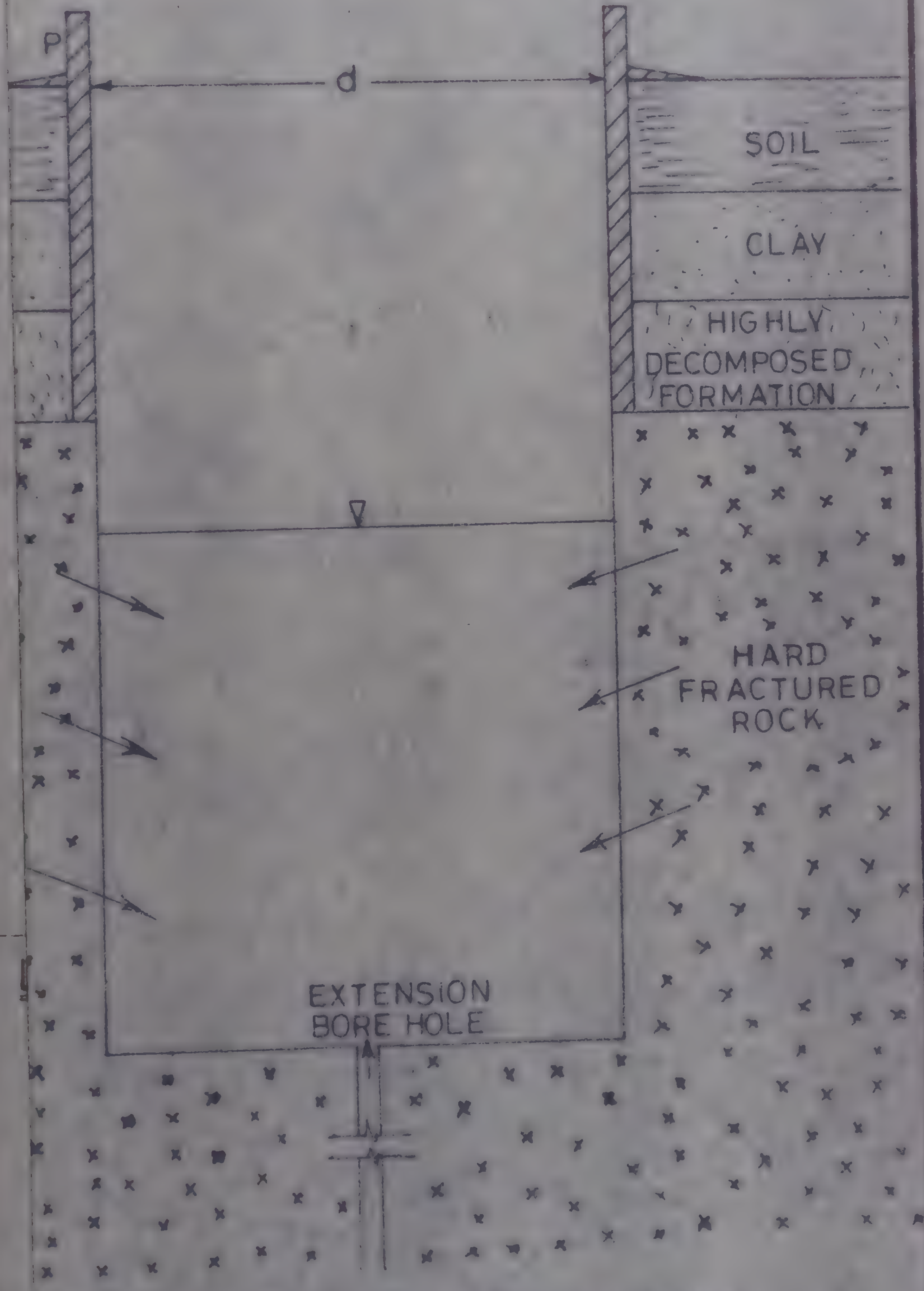
The syphon pipe should be of galvanized iron, the diameter depending upon the discharge of the spring and the difference between the delivery point and spring chamber.

Design of Open Wells In Plantations:

As has already been explained earlier open wells would be best suited for tapping ground water resources as these are capable of storing large supplies during a given recovery period and available supplies can be obtained at relatively small draw-downs (unlike borewells) in relatively short periods allowing for long non-pumping recuperation periods. They expose a greater surface area of the aquifer for infiltration. The most important zone is the weathered zone.

The size and depth of an open well will depend to a great extent on various factors like geological formations, structures, yield expected, water table fluctuations, etc. For purposes of Illustration the typical wells that could be sunk are shown in figure 17A to 17C. Although several different conditions may exist, basically the size and design would depend on the depth at which groundwater is encountered and on the quantity. Referring to figure 17 in the case of A, the water is encountered in highly weathered or decomposed formation which needs lining of the loose formations in the well upto the bottom. The diameter does not need to be more than 5-6 metres with depths of about 7.0 metres. In (B) the well taps a sandy formation

(C) WELL NEEDING PART LINING, MATERIALS AS IN (A)



FRACTURE FORMATION YIELDING

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which yields water. The lining here should be concrete rings above which stone/bricks in cement mortar could seal off the top soil zone as in A. Concrete rings facilitate construction as the sides of the well tend to cave in as excavation progresses necessiating simultaneous digging and lining which cannot be done in the case of brick or stone. The concrete rings also called pressure rings) sinks as digging progresses. The depth does not need to be as much as in (A), say 4.5 to 5.0 metres but the diameter should be large say 7-8 metres. In case of (C) the well taps groundwater in hard, fractured rocks, here the depth may have to be more but diameter will need to be controlled, to reduce cost which could be better utilised to penetrate depth. Thus a well of about 8-10 metres with diameters of about 6 metres would be necessary. Lining of stone/brick and cement would be needed only upto the level where hard rock is encountered. In all the three cases it is advisable to have a parapet wall 2 feet about ground level to act as a protective cover for direct inflow in times of flood or heavy rains since these wells would generally be quite close to surface drains or streams. It must be mentioned here that depths and diameters suggested are not absolute and have to be decided at site since conditions vary greatly.

In case of well which may have struck hard rock, horizontal drilling may enhance supplies from joints or fractures while vertical extension boreholes could further augment the supplies. This however will have to be decided depending upon the prospects. The lining in all cases must ensure that surface contamination is sealed out.

Parapets should have a platform of concrete around the well and sloping away from it. As far as possible the top should be covered by a proper gauze to keep out possible contamination by direct withdrawal or by entry into the well. In many localities however, the well maybe utilised by direct withdrawal

by rope and bucket. In such cases it is preferable to cover the top and provide a handpump thus eliminating possible contamination.

Open wells should be preferably rectangular rather than the traditionally accepted circular types. Larger cross sectional area will give more chance to cut across fracture zones and thereby increase recharge. This also provides larger storage. The longer side of the rectangular well should be normal to the trend of the fracture system to ensure better yield. Walls should also touch basement (fresh) rock as the top of the fresh rock is likely to be a more fruitful zone than the remaining part of the weathered zone.

Design of Borewells In Plantations:

Borewells are sunk when deep weathered zones with viable supplies are suspected and digging to such depths would be uneconomical or impractical. This would also be the case where fissured or fractured zones are expected at depths in hard rock.

Down the hole hammer drilling is commonly employed in hard rock terrains such as plantation areas and in fact most of Peninsular India. These are high speed drills using compressed air as the power driver. Some old fashioned calyx drills may also be used in water well drilling in hard rocks.

A typical borehole section is shown in figure 18. The depth is determined before drilling : generally it is about 45-60 metres (150-200) feet in granitic terrain or about 5 metres after striking a productive zone. Yields are expected to be varied but for a drinking water well a yield of 450 LPH would be considered successful in such terrains. The diameter is generally (101 - 152 mm) 4-6". The well should be as vertical to enable installationof a suitable lift system later on. In figure 18 it can be seen that a casing pipe has been inserted upto the point where loose or very soft formation ends. This saves the well from being filled up by caving

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TYPICAL BOREWELL DESIGN
(HARD ROCK AREAS)

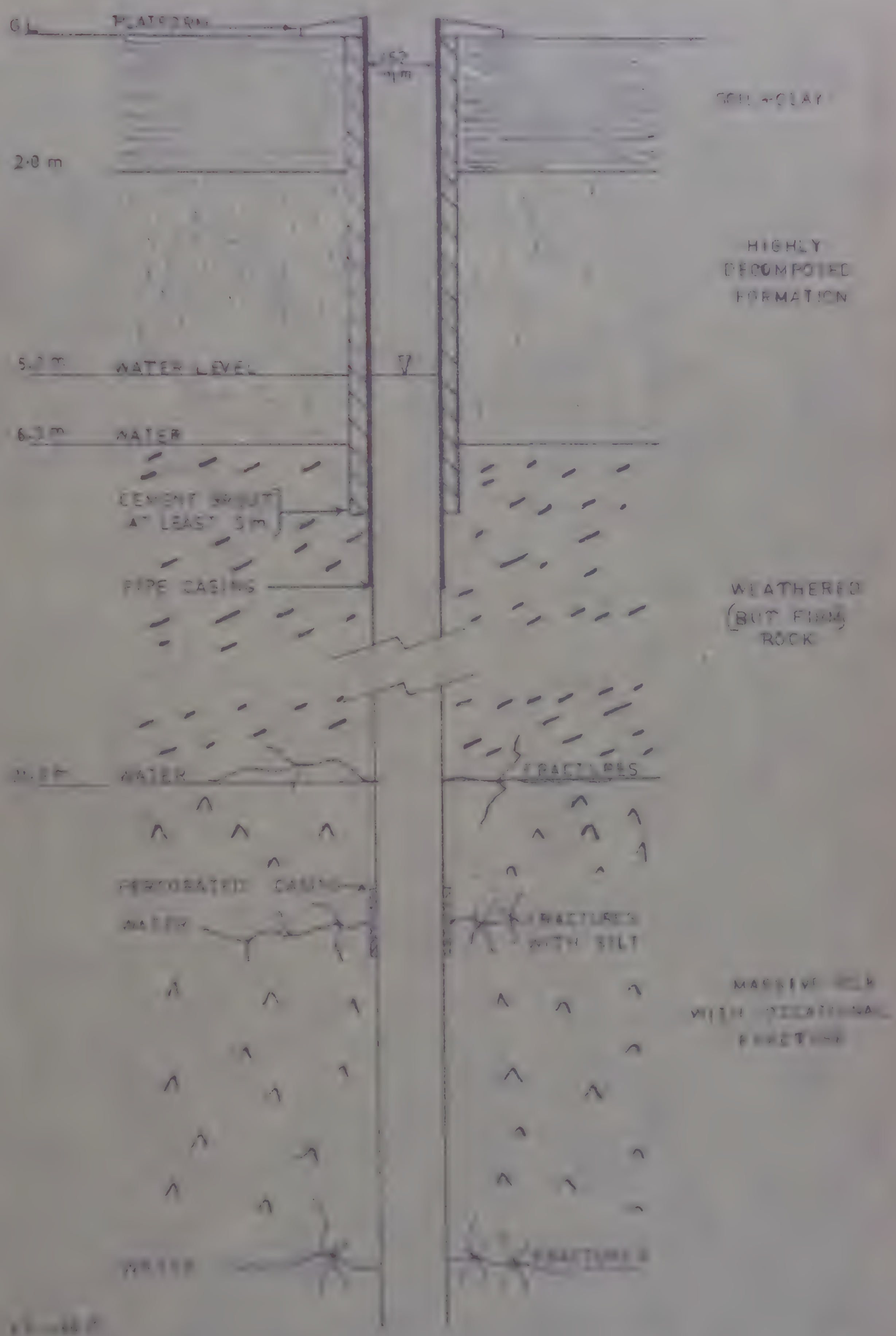


FIG. 8

in of the overburden (soil, clay, very soft and decomposed material). This is done before any further drilling which then continues unhindered till the well has been completed. There is no need for pipe casing once firm rock is struck. However, in some exceptionally rare cases silt enters the well from fracture zones in firm massive rock. In such cases the lift system should be operated by compressed air.

The top casing acts as a protection against contamination by infiltration from shallow zones, in addition to keeping the loose material in place. In cases where the well is likely to be polluted easily, a cement grouting may be provided as follows:

on completion of drilling of the overburden or atleast 5 metres depth the bore hole is widened or reamed (say from 6" to 12"). The casing 6" (I.D.) is then inserted and the space around (3") on all sides of the casing is packed with cement concrete. This acts as a seal to possible heavy filtration. In most cases this may not be necessary. A cement concrete platform around the well with a slope away from it is a must however, and the casing pipe should be protruding about 0.5 to 1 metres above ground level depending on the well site location.

The borehole must be flushed with air on completion of drilling until water emerging is clear of all sediment indicating that it is clean. In most operations it is possible to estimate the yield roughly using the compressed air and V-notch. Boreholes in hard rock terrains are generally expected to yield good quality water and may be safe for drinking direct from the well.

In the design of borewells and dug-wells, the increase in diameter does not necessarily mean a proportionate increase in yield can be expected. Table IV shows the relation between diameter and yield.

Lift from continuous surface flow: These are essentially from streams having a continuous minimum flow. There are two categ-

ories under this type:

(a) Lifts requiring no intake structure.

(b) Those requiring a weir to create a depth for pumping or ensure power head flow for hydrams.

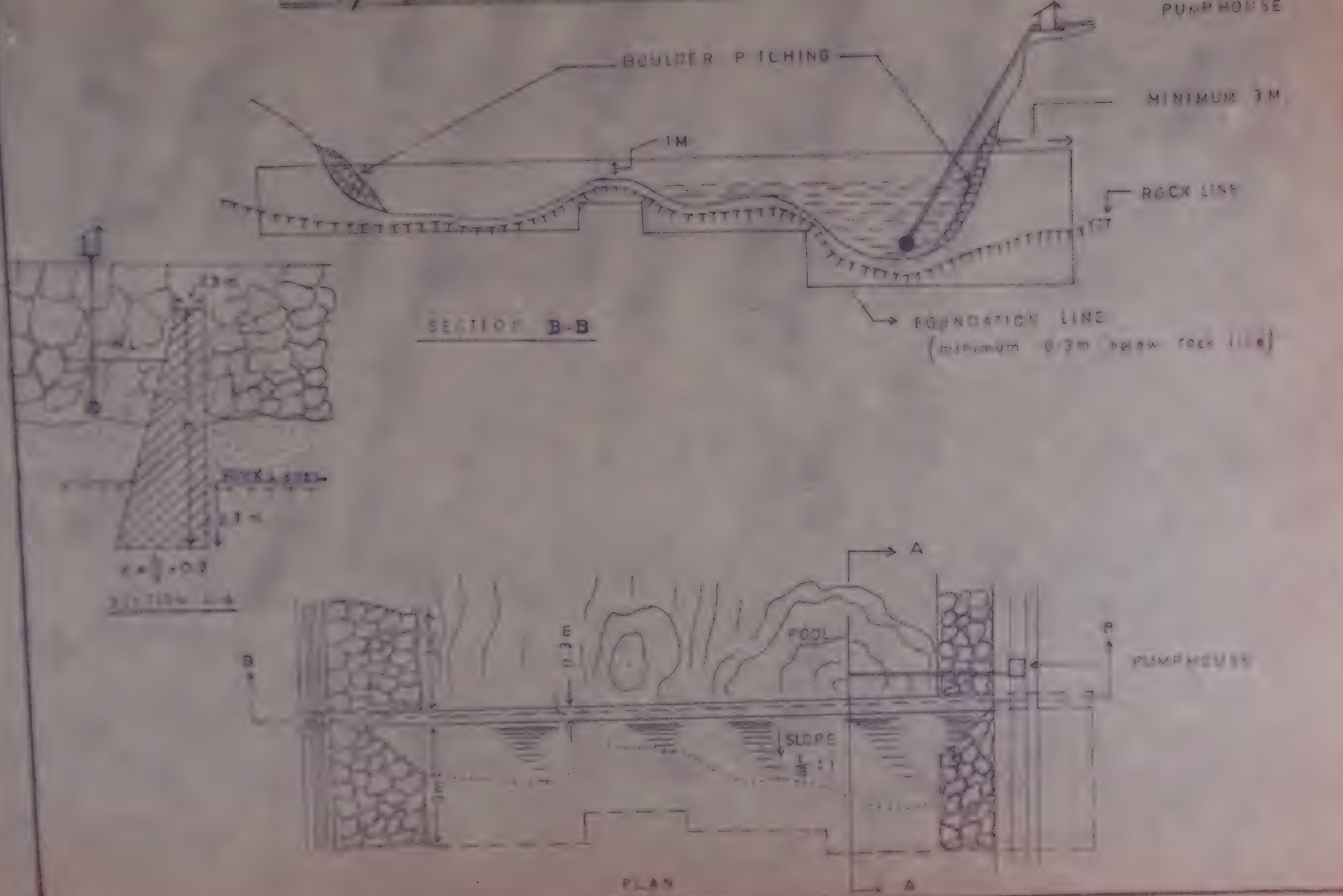
To avoid provision for storage at the intake the rate of extraction should be minimum with maximum pumping hours. A low pumping rate will also reduce the space and cost require-d for treatment and storage chambers. Hence the designs given below are based on the assumption that there is little or no storage required at the intake.

These are best located on the banks of rivers which have a deep, concentrated flow, or a natural sufficiently deep pond that collects all of the flow before allowing overflow close to the bank of pumping. In cases where such concentrated flow or pond is situated away from or near the opposite bank of the point of pumping, the suction pipe will have to be extended to extract water.

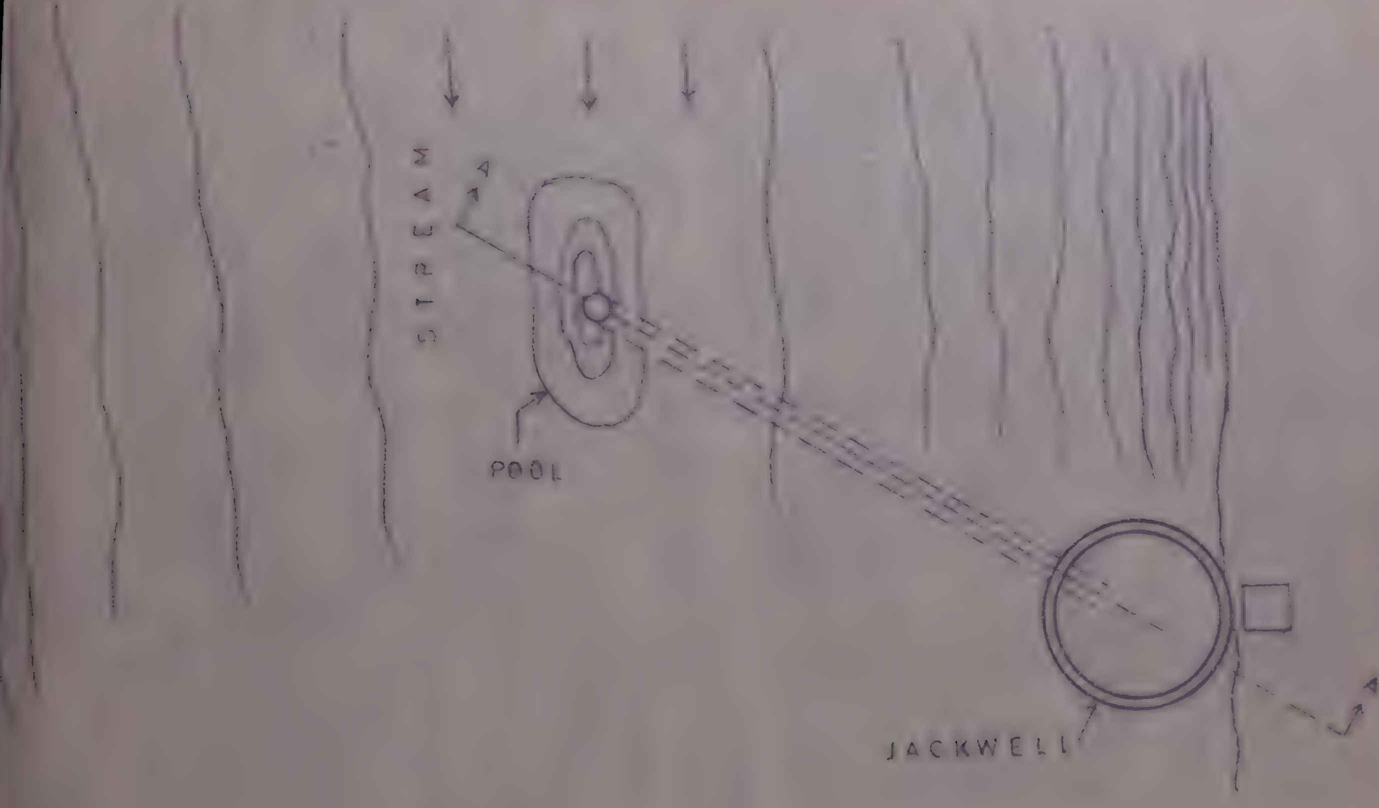
Weirs: Weirs are solid walls constructed across a stream or river, having a relatively lesser height. The purpose of a weir is not to create storage but to raise the level of water and thus facilitate convenient extraction by pumping or by gravity. In the present context of water supply in plantations, the height of the weir need not be more than 1 metre above the highest point of the stream bed. The top width should be 1m, the upstream side of the wall vertical and downstream side with a slope of 1 horizontal to three vertical. The foundation should be at least 0.3m keyed into the rock or in concrete 0.25m thick and 1.5m below the ground level at any point along the section, whichever may be the case. The length of the weir depends on the slope of the banks. For details refer to figure 19.

Jack wells/Collector wells: Jack wells are cylindrical shafts consisting of R.C.C. rings sunk in a stream or river bed. The bottommost ring should be keyed at least 0.2m to rock or rest on a concrete ring of 0.3m wide and 0.25m thick. The concre-te

FIG 19 DETAILS OF A WEIR



rings may be necessary if the rock is not struck within reasonable de-pth. The depth of the jack well is determined after assessing the site as detailed earlier, but normally need not exceed 8m, the diameter of the well (R.C.C.) rings should not exceed 5m and the minimum thickness of the concrete rings should be 150mm. A slotted pipe of 200 mm diameter is embedded in the concrete rings at a level of 3m above the bottom of the well. Since the jack well is usually located near the bank of the stream, the pipe should be extended across the stream to end in the nearest pool of water. The open end of the pipe should be covered by a wire mesh. The upward gradient of the inlet pipe (slotted pipe) away from the well is usually 1 in 100. The concrete rings should have 10 mm holes at a horizontal and at a vertical interval of 30 cm. A gravel packing 30 cm broad is provided around the concrete rings to prevent sand from entering into the well. For further de-tails please refer to figure 20.



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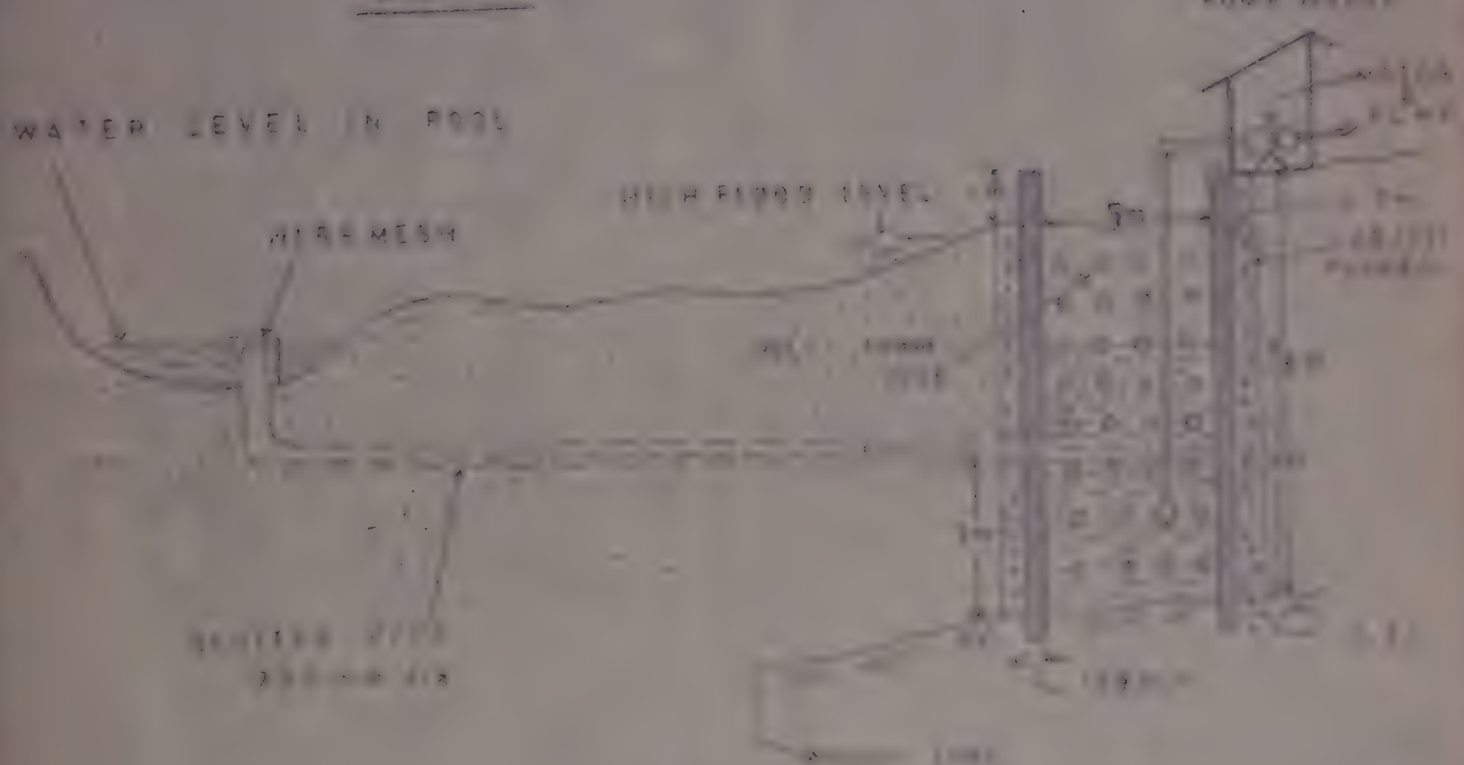


SECTION A-A

FIG 10 DETAILS OF A JACKWELL

Pumps for Water Supply:

The applicable types of pumps as far as water supply schemes in plantations are concerned, are:

(a) Reciprocating pumps
(b) Centrifugal pumps
(c) Turbine pumps
(d) Submerisble pumps
(e) Jet pumps (Ejecto pumps)
(f) Hydraulic Rams (Hydrams)

The other type of pumps like rotary pumps, air lift pumps, etc., find limited applicability in the present context. Since the details of construction, efficiency, formulae for discharge etc., vary for different manufacturers, such details for all displacement pumps (a through e above) are supplied by the manufacturer. Hence only the general principles involved are discussed.

(a) Reciprocating pumps: the principle involves drawing in water through a suction pipe by a plunger moving up and down, or forward and backward in a chamber, and thus creating a vacuum. The water is pushed up simulataneously with the help of non-return valves. The major advantage of reciprocating pumps in estates, is that, they can pump small amounts of water to very large heights with a lesser power (HP) compared to centrifugal pumps. For details refer figure 22.

(b) Centrifugal pumps: A centrifugal pump is widely used in water supply, irrigation, factories, etc. The principle involved is that an impeller rotating inside a close fitting case draws in any given liquid at the centre and by the virtue of centrifugal force, pumps it out through an opening at the side of the casing. The high velocity produced by the impeller is partially transformed into pressure head. The details of a centrifugal pump is shown in figure 23 (A) a nd 22 (B)

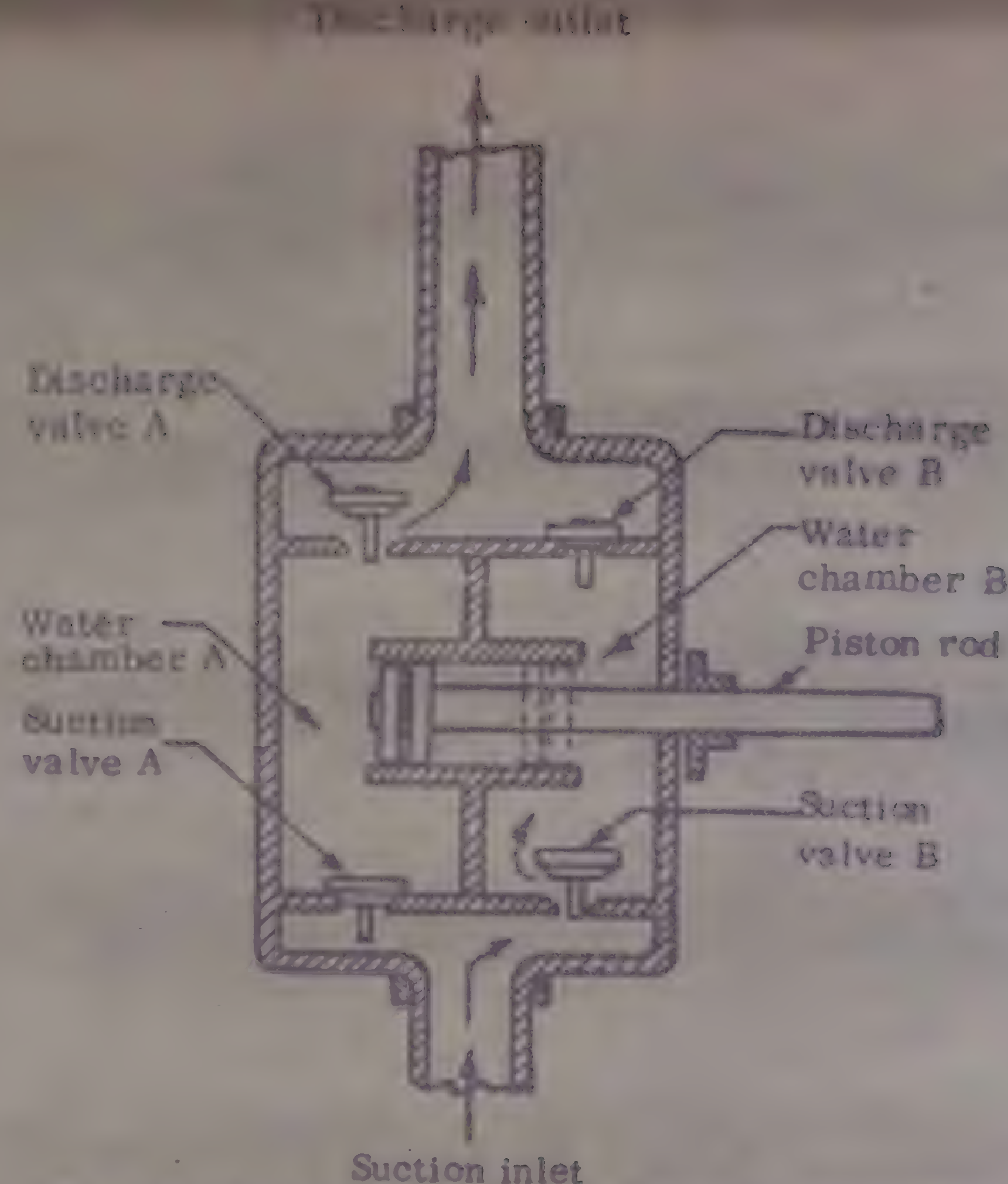


Fig. 2.1. Sketch illustrating the priniciple of working of a double acting reciprocating pump.

Fig. 22A CENTRIFUGAL PUMPS : DIRECTLY CONNECTED TO POWER UNIT (I) AND WITH BELT-DRIVE (II)

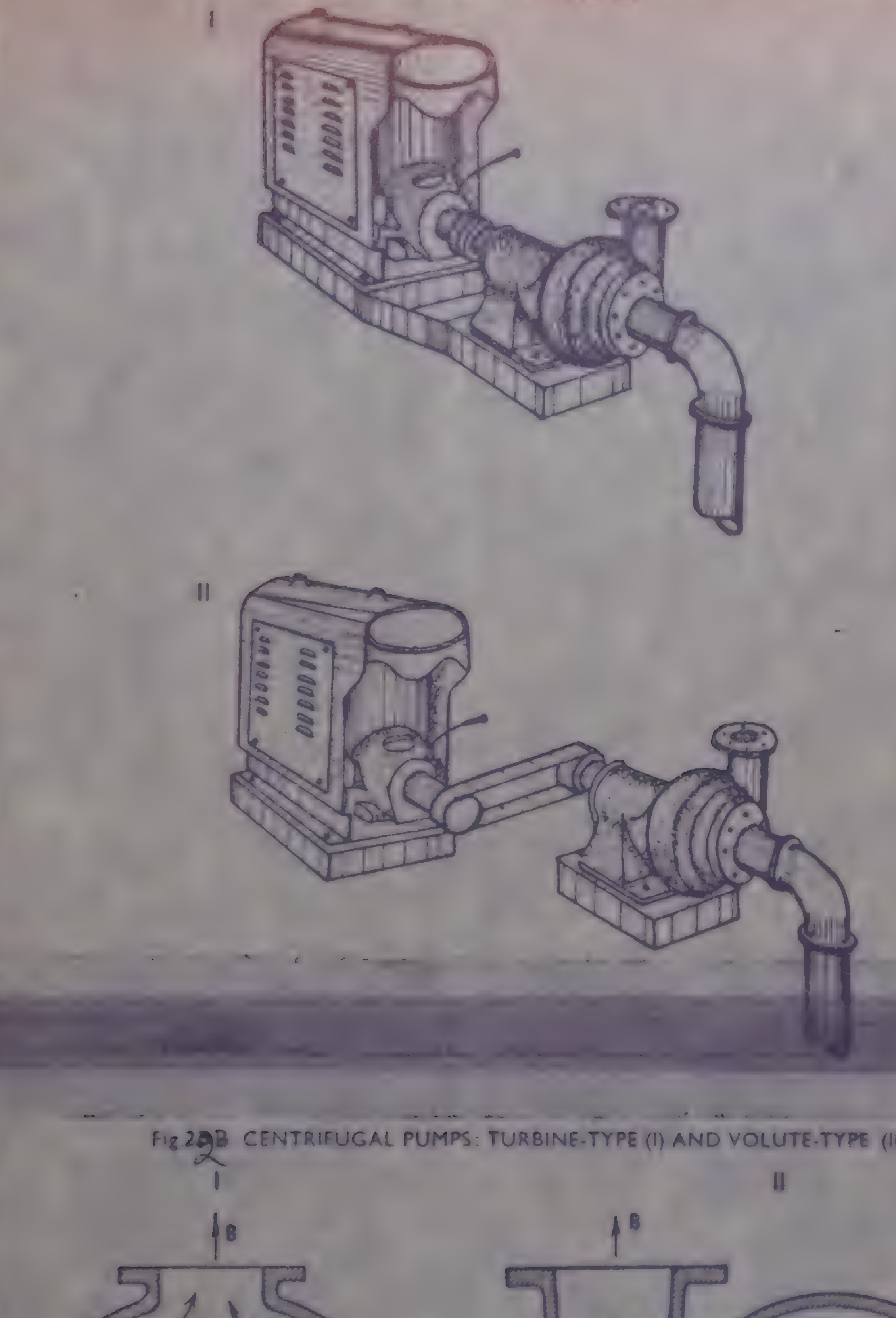
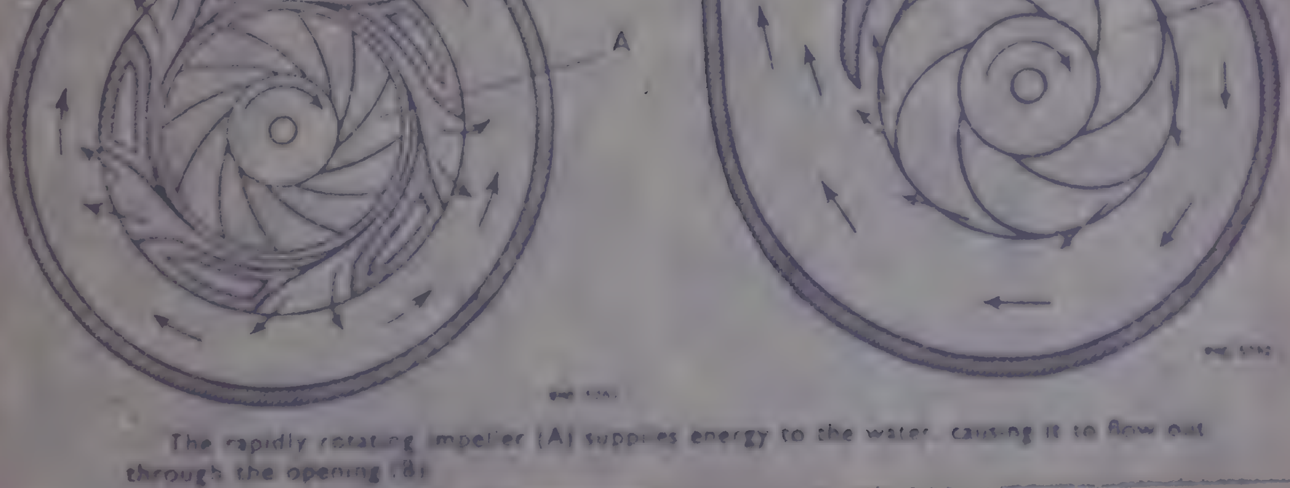


Fig. 22B CENTRIFUGAL PUMPS: TURBINE-TYPE (I) AND VOLUTE-TYPE (II)



The rapidly rotating impeller (A) supplies energy to the water, causing it to flow out through the opening (B).

(c) Vertical turbine pumps: These are impeller type vertical centrifugal pumps with a surrounding discharge pipe and a drive shaft in the middle. They are (the impellers and the suction casing) necessarily submerged and driven by the motor and engine. The principle is same as that of a centrifugal pump excepting that the impellers and the impeller bowl chambers are submerged in water. Figure 23 gives details of a typical turbine.

(d) Submerisble pumps: These are vertical turbine pumps with both the prime mover (electric motor) and the impeller chamber submerged in water. The entire pump comes in a housing consisting of the motor and the impellers, which is connected to discharge pipes and lowered into the bore-well. Figure 24 shows the detailed section of a submersible pump.

(e) Ejecto pumps: Ejecto or jet pumps are operated with the help of a high pressure water jet sent through a pipe (pressure) which pushes up water through a second (delivery) pipe. This type of pumps find applicability in low yielding bore wells and requiring heads upto 30 metres. Figure 25 gives details of construction and installation.

(f) Hydraulic Ram: The Hydraulic ram though not very popular due to the lack of the conditions required for its functioning, is an impulse pump being self-operated by the water hammer intentionally produced. The conditions required for its operation are:

(i) The continuous available supply should be 3 to 30 times more than the supply required.

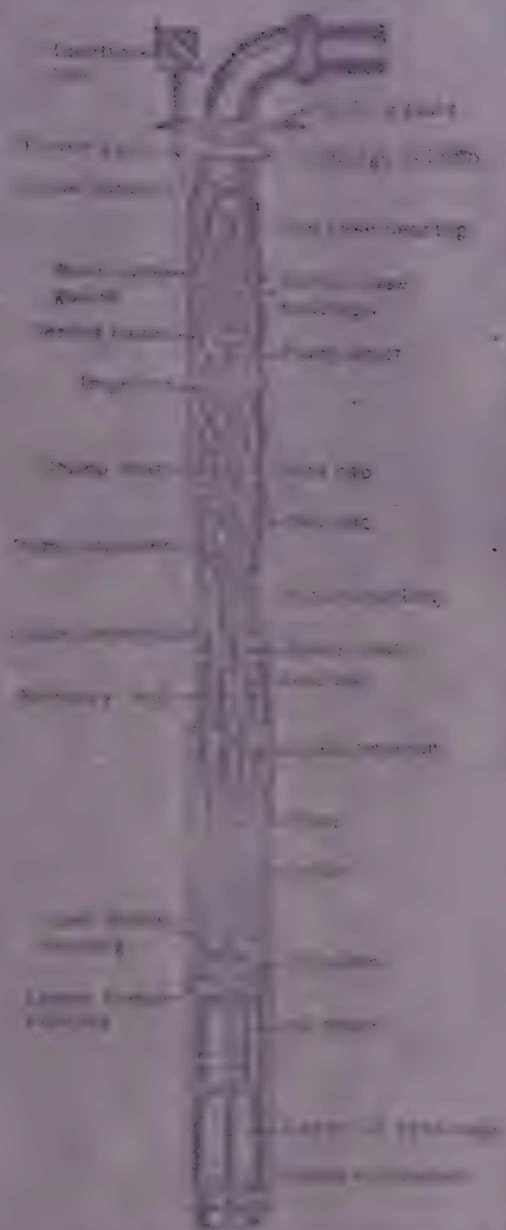
(ii) There should be a sufficient fall to create velocity in power pipes and hence water hammer.

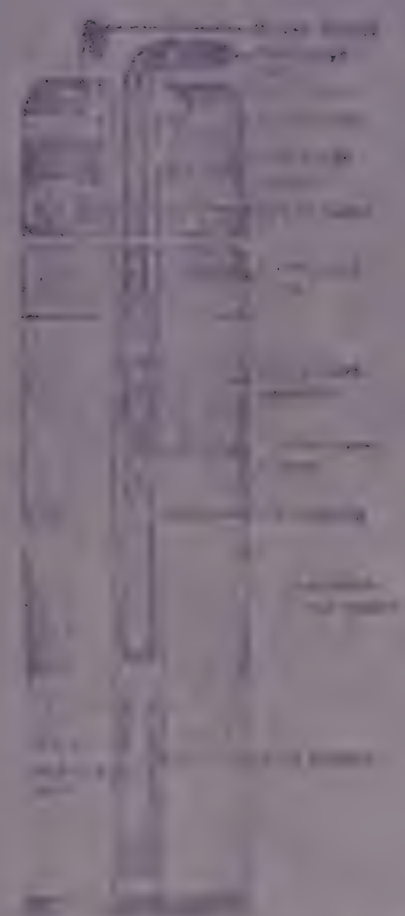
The force of water is captured in a chamber (see figure 26) where air is compressed and released when the same compressed

Fig. 13 SECTION OF TYPICAL DEEP-WELL TURBINE PUMP

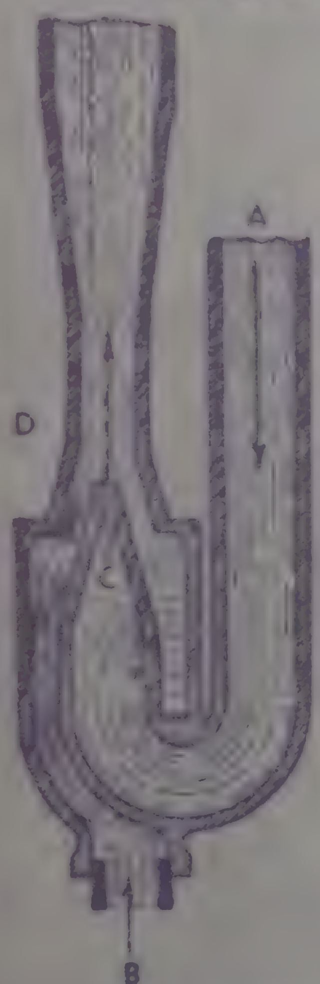
A — Impeller
B — Stage
C — Shaft
D — Strainer

In the turbine pump, water is lifted from one stage (B) to stage by each impeller (A) being rotated by shaft (C), which is attached to power unit. The number of stages and the design of the impeller vary with the pumping requirement. This pump is highly efficient and, when properly operated and maintained, gives long service.



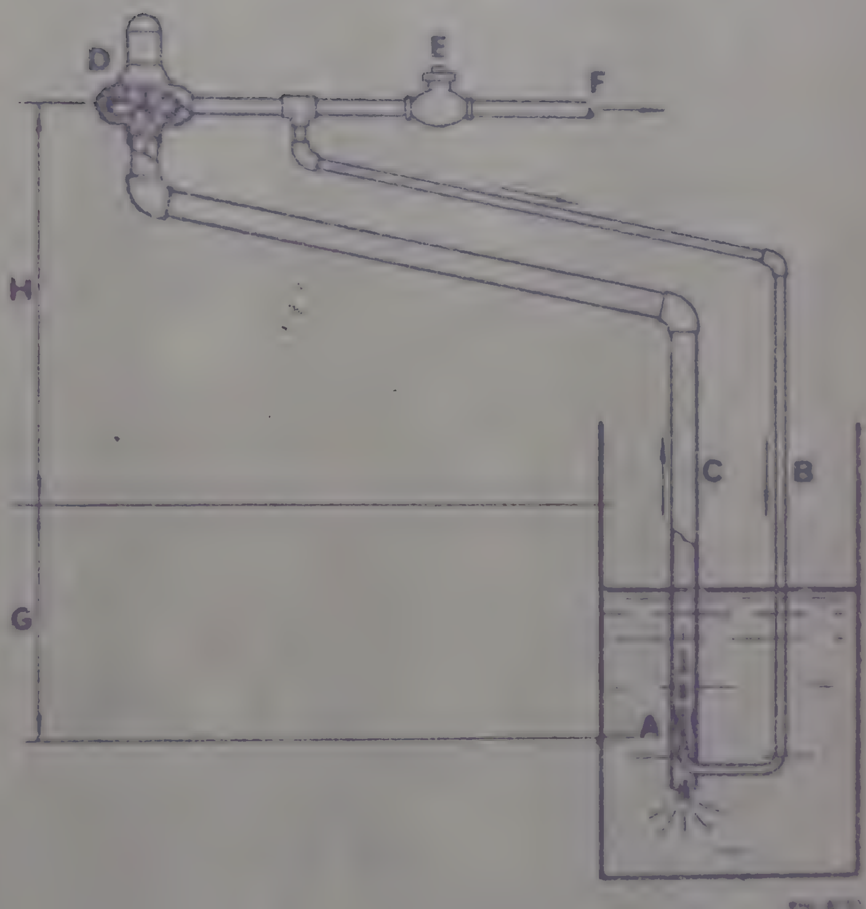


JET PUMP



A = Water being returned from pump above
B = Water from well being sucked up into throat (D) by high-velocity discharge (C)

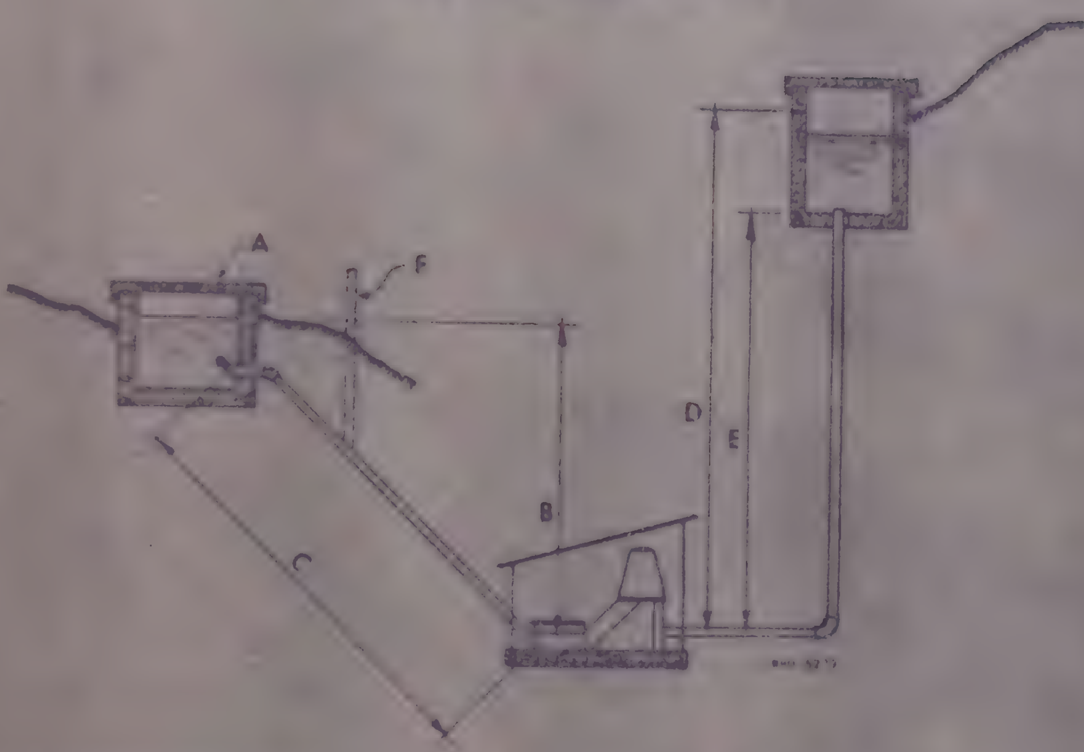
TYPICAL INSTALLATION OF JET PUMP



A = Jet assembly
B = Water line from pump to nozzle
C = Rising water
D = Centrifugal pump
E = Pressure-regulating valve
F = Discharge pipe
G = Height of water pushed by jet
H = Suction by centrifugal pump (about 4.5-6 m. or 15-20 ft)

FIG. 25 DETAILS OF JET PUMP

Fig 26 HYDRAULIC RAM



A — Supply—litres minute
B — Difference in elevation between ram and supply-power head
C — Length of drive pipe
D — Difference in elevation between ram and highest point to which water is to be elevated—pumping head
E — Total length of supply pipe
F — Stand pipe, necessary in case of exceeding y long drive pipe

air expands, pumping a part of the water to a higher elevation than the point from which it originally came. The water not pushed up escapes through an escape valve and is wasted unless subsequently used. This process can go on continuously as long as there is a minimum flow at the source. When the source flow reduces, the power pipes do not flow full and the ram stops operating. The following Table IX represents the amount of water that can be pumped with various heads for the different amounts of power water:

TABLE IX

Ratio of pumping head to power head	2	3	4	5	6	7	8	9	10	12	15	20
Delivery in gal per day for each gal per minute of power water	540	345	240	192	160	137	120	107	96	80	64	43

The heads are inclusive of the frictional losses in pipes and in the ram.

The advantage of hydraulic ram over other pumps are many. The initial cost is a minimum and running/maintainence cost is almost nil. Except that the ratio of pumping discharge to the available discharge is comparitively low, Hydraulic rams are best suited for plantations.

Hydraulic Parameters: Those faced with pump problems and pump selection come across several terms, a clear cut understanding by physical concepts of which is essential for tackling the problems. These are explained below:

Head: Hydraulically the head is the difference in elevation between two points in liquid bodies. For instance a difference in level between the water surface in a well and the surface

FIG. 2.7

SCHEMATIC REPRESENTATION OF DIFFERENT TYPES OF HYDRAULIC HEADS

water in an overhead tank is called a head. Figure 2 illustrates different types of heads.

Stat ic head: This is the difference in elevation between two static liquid surfaces.

Static Suction Lift: This term refers to centrifugal and reciprocating pumps and is the difference in elevation between the centre of the pump and the level at the source. The maximum possible suction lift theoretically is 10m at the sea level but for all practical purposes this should not exceed 6.5m.

Total Suction Lift: is the sum of the frictional losses in the suction pipe and static suction lift.

Static Discharge head : This is the difference in elevation between the centre of the pump and the discharge level.

Total Discharge Head: This is the sum of the static discharge head and the frictional losses in the discharge pipe.

Velocity in pipes: This is the speed of the water in the direction of flow. This can be obtained by dividing the discharge in the respective unit by the area of the pipe section. The unit is metre per second.

Friction head: This is the equivalent head expressed in metres of water required to overcome the resistance created by the pipe surface during conveyance.

Design Discharge: This is the rate of flow for which pumps have to be selected and is in fact based on the extractable amount from each source determined as discussed earlier.

The design discharge for pumps should not exceed the minimum extractable amount from each source. In case the minimum daily extractable amount from each source is more than the daily requirement of consumer points under it then the design discharge should be obtained by dividing the daily requirement by the daily pumping hours.

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Daily Pumping hours: For water supply schemes in plantations, more daily pumping hours with less discharge rate is preferable to short duration pumping with high capacity pumps for the following reasons:

(a) A high rate of discharge would need large diameter pipes for conveyance, large capacity treatment plants and storage structures at the intake and consumer points.
(b) With less than half the cost of a single large capacity pumping unit one can have small capacity pumping unit if the daily pumping hours are doubled.

It is suggested that the daily pumping hours should be kept in the range of 12 to 20 depending upon the source and requirement.

Number of pumping units at each source: Though theoretically it may be advisable to have one standby pumping unit for each source in practice, however, water supply for plantations where the demand at scatterred consumer points are met from different sources, a standby may not be advisable. The life of the pumping unit can be prolonged by giving about an hours rest between pumping sessions. Even in case of breakdowns this will affect only part of the community. However if the whole estate is served by a single source, it is imperative to keep one standby pumping unit.

Selection of pumps:

It is difficult to suggest criteria to be adopted for selection of water supply pumps as many factors influence the design.

However some guide lines are given below:

(a) It is not necessary to calculate exactly the H.P., the r.p.ms impeller diameter stages of the pump, types of pump etc.,.Each manufacturer will have their own specifications and when contacted with the necessary data would supply the correct combination.

(B) The following are the basic data that should be obtained before contacting the dealers of different pumps/motors:

- The design discharge
- The total static head
- Maximum fluctuation in water level at the source
- Number of pumping hours
- Frictional losses in the suction and delivery pressure pipes, valves, junctions, bends, etc.

(c) A reciprocating pump would be the best suited for surface water sources and open wells when the discharge is very low and the head is very high as in the case of estates. However, for higher discharges with lower heads, centrifugal pumps may be considered. If the seasonal fluctuation of the water level is high, then there should be provisions to shift and install the pumping units at different stages to facilitate easy suction. In open wells and jack wells this is done by constructing girders at different levels and is not very difficult if the prime mover is an electric motor, directly coupled to the pump.

(d) Turbine pumps should be preferred:

- When the water level fluctuation is quite high and frequent.
- If the discharge required is very high.
- For high yielding borewells.

(e) Submersible pumps are best suited:

- where electric supply is available
- for low to medium yielding borewells with low to medium head.

(f) Hydrams are best suited:

- for streams in hilly terrains
- where there is a minimum continuous flow which is several times more than the minimum requirement.
- where the mountain stream has enormous torrential flow during rainy season, a part of which is proposed to be conserved (for use in lean periods) at a higher altitude

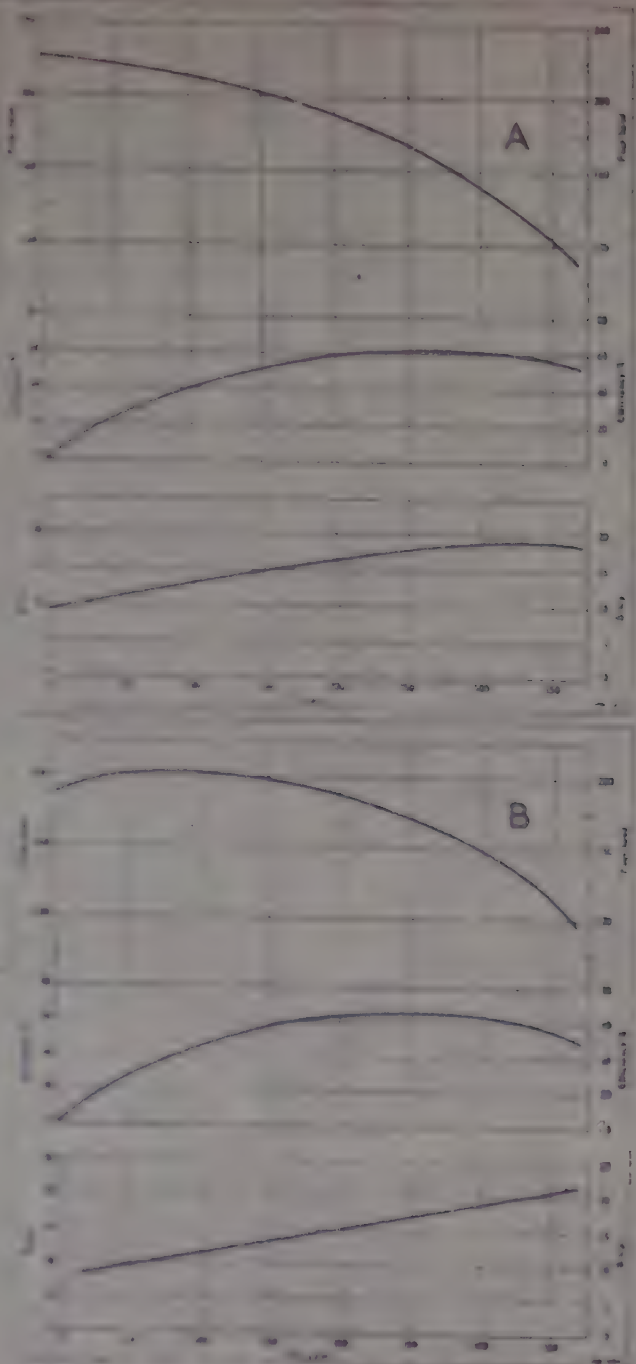
in underground reservoirs.

(g) An ejecto pump is best suited for low yielding bore wells and requiring low heads.

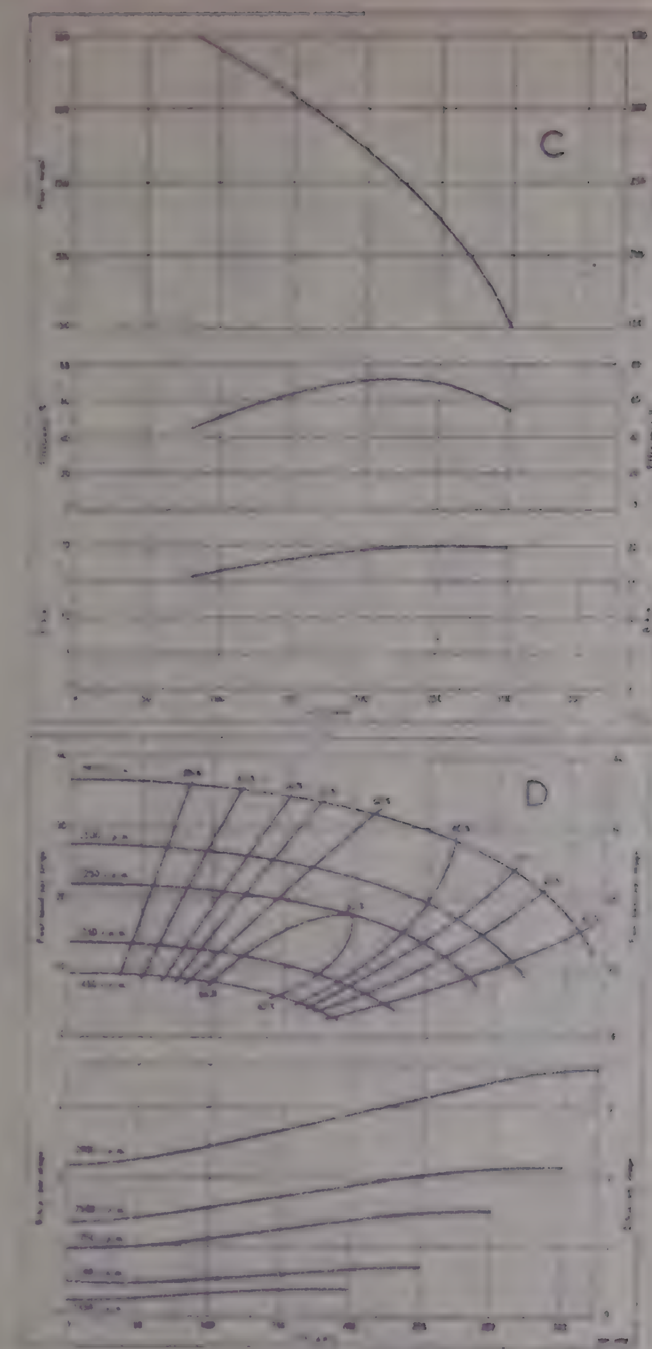
The efficiency chart for turbine pumps and centrifugal pumps are shown in figure 28 and 29 respectively. The affect of variation of suction lift on the efficiency of a centrifugal pump is represented in figure 30. It may be appreciated that the guidelines mentioned above are general and several other factors like mechanical and chemical quality of water, availability of after sales service, availability of power supply and cost factor also influence selection of a pumping unit.

Pump house: These are small cabins usually made of the simplest, cheapest and of the most durable construction material, to protect pumps and motor/engine installations from sun, rain, etc., as well as from theft. The walls could be of stone, brick, G.I. sheets or asbestos sheets. The roof could be of G.I. sheets or asbestos sheets. The size should be a minimum of 2.5m x 2.5m x 2.5m. The pump house should be located as close to the source (usually not more than 2m) as possible. The floor level of the pump house should be at least 1m above the highest flood level in the stream. In case of centrifugal pumps and reciprocating pumps, the floor level should be a maximum of 6m above the lowest water level in the source.

8 TYPICAL CHARACTERISTICS OF DEEP-WELL TURBINE PUMPS



A, B = 3" multistage centrifugal pump, 1450 r.p.m.



C = 1½" five-stage submersible pump, 2900 r.p.m.
D = 6" deep-well turbine pump

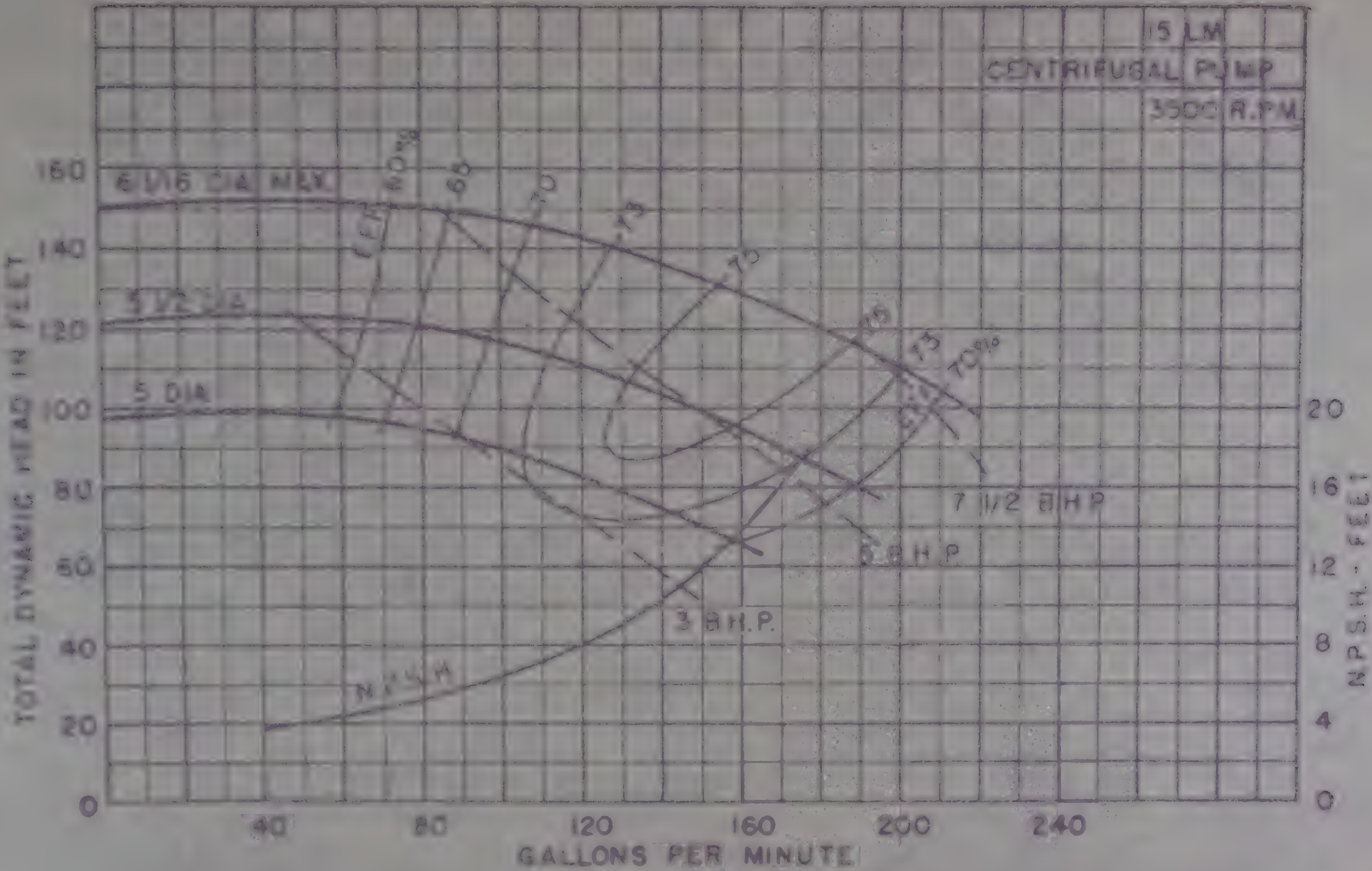
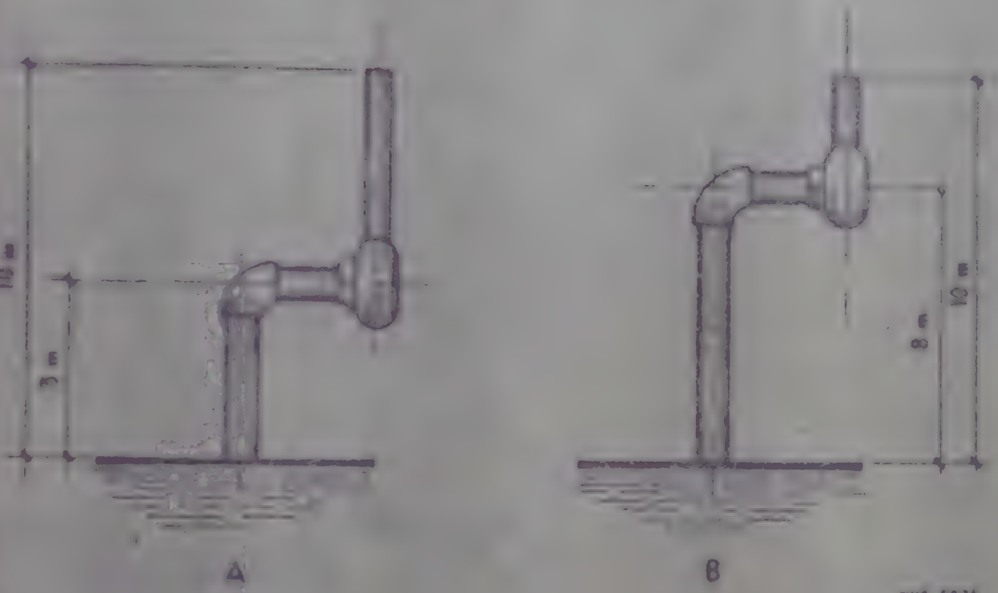
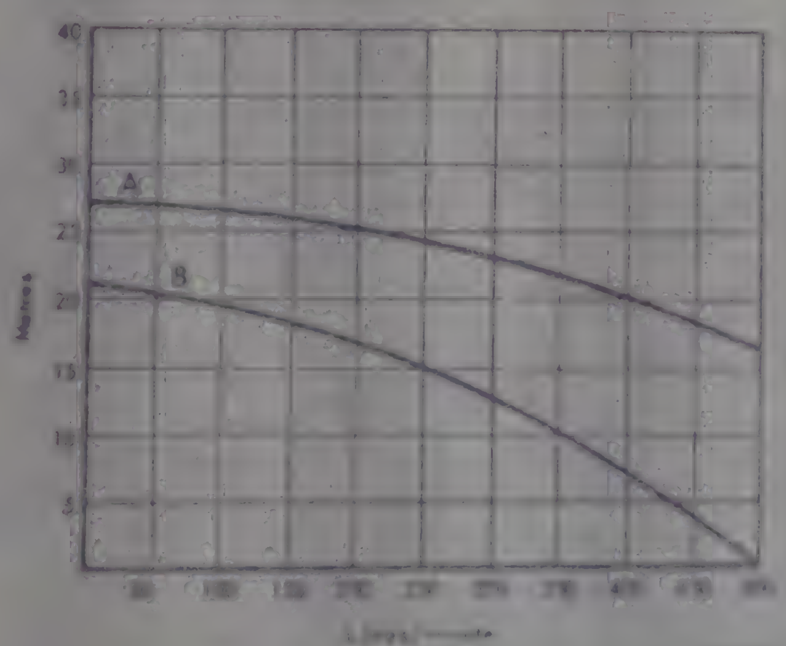


FIG. 29 TYPICAL EFFICIENCY CHART FOR CENTRIFUGAL PUMP

Fig. 31 TWO PUMPING SITUATIONS, SHOWING CENTRIFUGAL PUMP CAPACITY CURVES





A — Performance curve when pumping with low suction lift
B — Performance curve when pumping with high suction lift

Conveyance System:

The Conveyance system in a water supply scheme includes the pressure pipes transporting water from the source to the treatment plant and storage chamber, gravity pipes connecting the intermediatory delivery/storage chamber and the consumer points, and the auxillary structures like, pressure release valves, wheel valves, non-return valves etc. A proper design and correct selection of modes and types of these components is most essential to ensure an efficient and economical conveyance system. The only mode of conveyance applicable to water supply scheme is pipelines.

Different types of pipes: The three different types of pipes that can be considered for adaptation in plantation water supply are:

(a) Galvanized iron (G.I.) pipes
(b) Super pressure resistant synthetic pipes being developed and introduced into market (HDPE).
(c) Aluminium pipes.

G.I. pipes: G.I. pipes have been successfully used in water supply schemes for the past several decades. The major advantages of this type of pipes are:

- can be used to carry discharges at extremely high pressure.
- need not be buried underground as they cannot be damaged easily by mischievous elements.
- not affected much by the change in atmospheric temperature.
- relatively longer life period.

But G.I. pipes have been found to be subject to corrosion while carrying raw water, resulting in increase in the roughness of the surface thus adding to the frictional losses and clogging easily due to incrustations.

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Synthetic pipes: Recently several types of pipes of different specifications made of synthetic materials developed from petro-chemicals are being introduced into the market. It has also been observed that, synthetic pipes have the following advantages over other kinds of pipes.

- Very low frictional loss (70% of that of G.I. pipes)
- Easy handling.
- Low cost of transportation and installation.
- Resistance to any kind of corrosion.

However, there has not been extensive use made of such pipes in India, and their availability, durability and versatality have yet to be known from the manufacturers.

(c) Aluminium pipes: Aluminium pipes which are commonly used for sprinkle irrigation also find applicability in water supply systems. They have the following advantages:

- very light and hence easily handled and transported.
- more resistance to corrosion and less frictional losses compared to G.I. pipes.
- Capacity to sustain very high pressure.

However, aluminium pipes need special chemical coating to prevent electrolytic pitting especially when buried underground.

Criteria for selecting the types of pipelines: Depending on the pressure and design discharge for each part of the conveyance system, the following guide lines should be kept in mind while selecting the type of pipe lines.

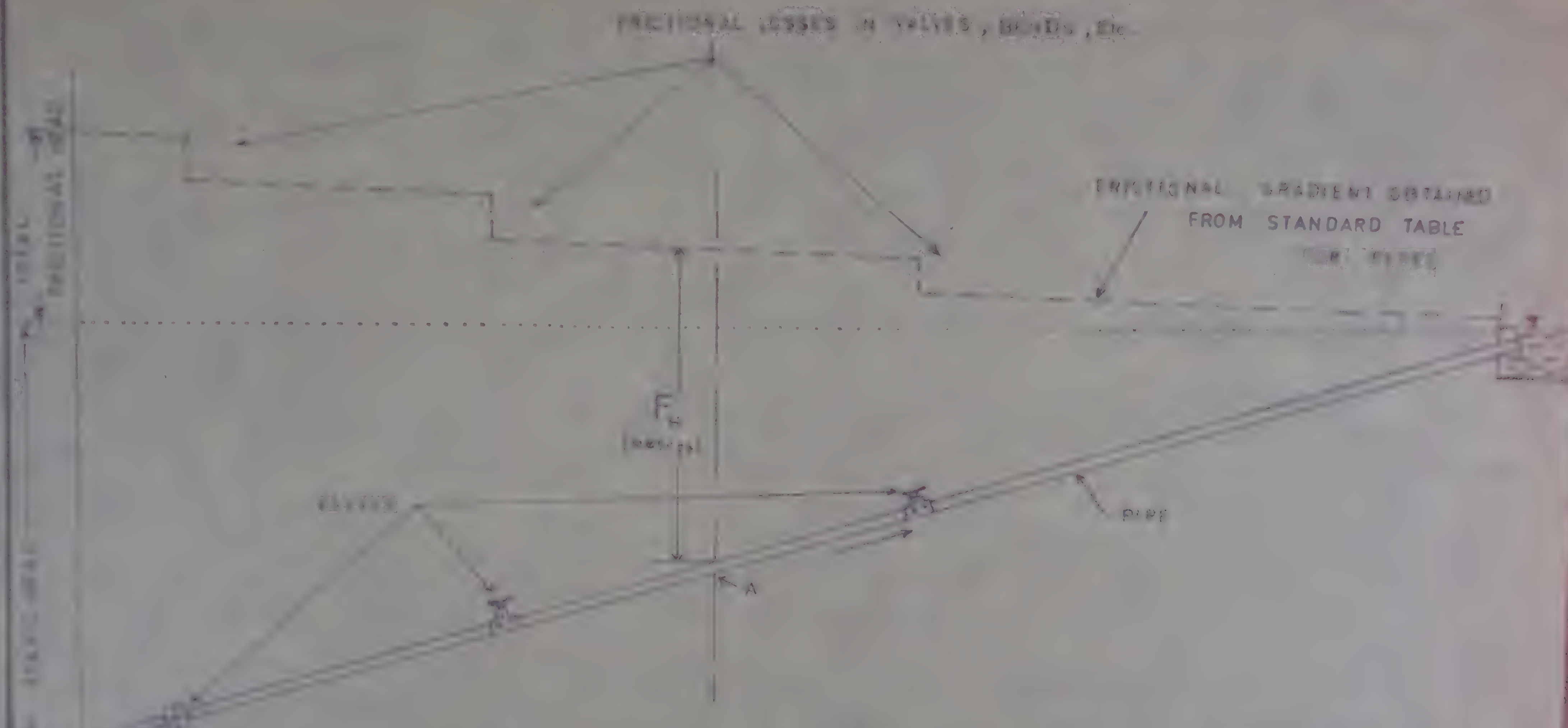
- Synthetic pipes of pproven quality for stretches of the conveyance system with low to medium pressures.
- G.I. pipes or aluminium pipes for certain stretches of the conveyance system where there is a possibility of higher pressure being developed.

- G.I. pipes for inflow and delivery lines of hydrams to resist the water hammer force.

Design of Pipelines: Having selected the types of pipelines for different reaches of the conveyance system, and with the help of the necessary data of discharge at each stretch, their respective lengths etc., the design of the pipelines involves selecting an optimum diameter which in the present context means minimum diameter with reasonable frictional losses per unit length. These losses for different discharges and different diameters for G.I. pipes are given in Table X. A similar table for synthetic pipes is not available though it is expected that their respective frictional losses are several times lesser than that of G.I. pipes. The total frictional loss for each stretch is obtained by multiplying the unit frictional loss by the length of that stretch. Normally the diameter is selected so that the frictional loss per 100 metre length does not exceed 1 metre. The pressure at any point along the pipeline is obtained by reading from the frictional gradient diagram (Figure 31).

Frictional losses in auxillary structures: - The frictional losses in structures like foot valves, non return valves, and bends also add to the total frictional loss of the conveyance system. The figure 32 presents a nomogram to help determine losses in such components.

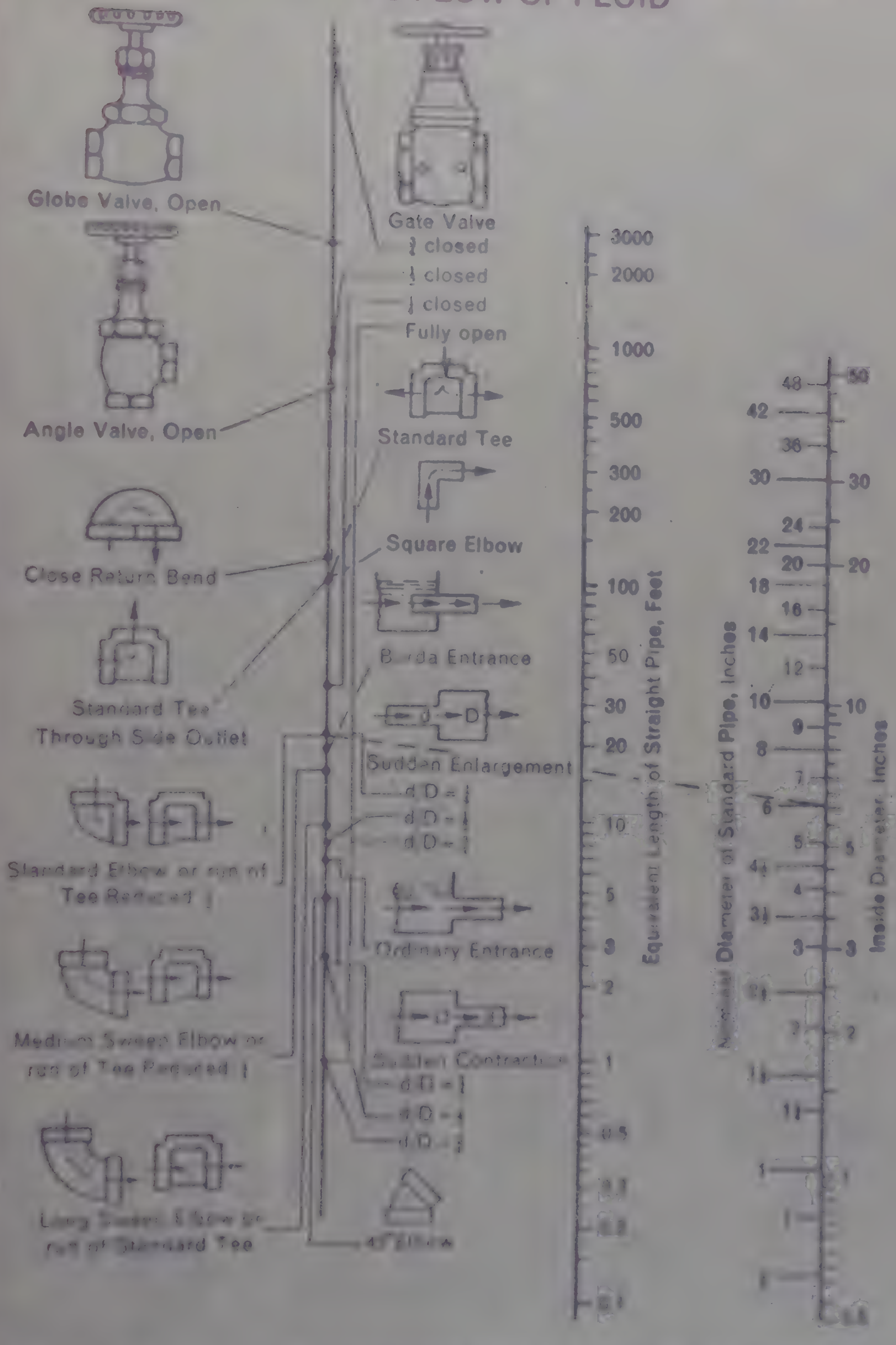
Pressure release valves: Each pipe is designed to sustain a prescribed pressure. The pressure in the pipe should not be allowed to cross this limit. To ensure this, pressure release valves are provided along the pipe line at intervals of half the total length or at every 100 metres which ever is less. The valve is controlled by a floating ball which raises and releases the excess pressure that may build up. The figure 33 shows a cross sectional view of a pressure release valve. The pressure release valves are a must in case of delivery and power pipes of a hydram.



ABSOLUTE PRESSURE AT A = [FH × 100 × density of water in Kg/Lt − Atmospheric pressure in Kg/c^2]

FIG. 4 Frictional gradient of pipeline

RESISTANCE OF VALVES AND FITTINGS TO FLOW OF FLUID



Example: To find resistance of 6" Standard Elbow in equivalent pipe length. Place a straight edge on the point representing Standard Elbow on the fittings chart scale. It cuts the Equivalent Length of Straight Pipe at 16 feet point. Therefore, friction in 6" Standard Elbow would be the same as you would get in 16 feet of 6" Straight Pipe for any quantity of flow.

Please note that for sudden enlargement or sudden contractions, the value of smaller diameter (d) should be used on pipe size scale.

Fig 32

Fig. 33 An air release valve used with high pressure pipelines.

TABLE X. Head loss in metres due to friction in galvanized iron pipes per 100 metres of pipe length.

Discharge litres/sec	Pipe diameter, centimetres										
	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	25.0	30.0
1.0	3.7	1.1	0.43	-	-	-	-	-	-	-	-
1.2	5.0	1.6	0.58	0.27	-	-	-	-	-	-	-
1.4	7.3	2.2	0.83	0.37	-	-	-	-	-	-	-
1.6	9.2	2.8	1.10	0.50	0.23	-	-	-	-	-	-
1.8	11.8	3.7	1.40	0.62	0.29	-	-	-	-	-	-
2.0	15.5	4.5	1.70	0.73	0.37	-	-	-	-	-	-
2.2	16.2	5.2	2.15	0.90	0.44	-	-	-	-	-	-
2.4	20.5	6.4	2.50	1.07	0.52	0.16	-	-	-	-	-
2.6	23.5	7.5	2.90	1.27	0.62	0.18	-	-	-	-	-
2.8	27.5	8.7	3.30	1.47	0.70	0.22	-	-	-	-	-
3.0	32.0	10.0	3.80	1.68	0.83	0.25	-	-	-	-	-
3.5	42.5	13.5	5.30	2.30	1.10	0.33	-	-	-	-	-
4.0	56.0	17.5	7.30	3.00	1.50	0.45	0.13	-	-	-	-
4.5	71.5	22.5	8.80	3.80	1.85	0.55	0.17	-	-	-	-
5.0	87.0	28.0	10.80	4.70	2.30	0.68	0.22	-	-	-	-
5.5	-	33.0	12.40	5.70	2.70	0.83	0.26	0.095	-	-	-
6.0	-	40.0	15.50	6.80	3.20	0.96	0.32	0.118	-	-	-
6.5	-	47.0	18.30	8.00	3.80	1.15	0.36	0.140	-	-	-
7.0	-	54.0	21.50	9.30	4.50	1.30	0.42	0.17	-	-	-
7.5	-	62.0	24.00	10.60	5.20	1.50	0.47	0.18	-	-	-
8.0	-	70.0	28.00	11.60	6.00	1.80	0.55	0.21	-	-	-
8.5	-	80.0	31.00	13.30	6.80	2.00	0.62	0.23	-	-	-
9.0	-	90.0	36.00	15.00	7.50	2.20	0.68	0.27	-	-	-
9.5	-	100.0	38.00	17.00	8.30	2.50	0.76	0.29	-	-	-
10	-	-	43.00	19.00	9.40	2.80	0.85	0.32	0.065	-	-

Discharge litres/sec	4.0	5.0	6.0	7.0	[illegible].0	10.0	12.5	15.0	20.0	25.0	30.0
12	-	-	63.00	27.00	13.00	4.00	1.23	0.47	0.10	-	-
14	-	-	86.00	37.00	18.00	5.50	1.65	0.63	0.13	-	-
16	-	-	-	47.00	23.00	7.20	2.20	0.83	0.17	0.055	-
18	-	-	-	60.00	30.00	9.00	2.80	1.05	0.22	0.068	-
20	-	-	-	72.00	37.00	11.00	3.30	1.30	0.27	0.080	-
22	-	-	-	86.00	45.00	13.50	4.10	1.60	0.33	0.100	0.038
24	-	-	-	-	52.00	16.00	4.80	1.90	0.40	0.120	0.045
26	-	-	-	-	62.00	18.70	5.60	2.20	0.47	0.145	0.055
28	-	-	-	-	72.00	22.00	5.60	2.50	0.55	0.165	0.060
30	-	-	-	-	80.00	25.00	7.50	2.80	0.63	0.19	0.070
35	-	-	-	-	-	33.00	10.30	4.00	0.85	0.27	0.095
40	-	-	-	-	-	45.00	13.50	5.20	1.10	0.33	0.128
45	-	-	-	-	-	56.00	17.00	6.50	1.40	0.43	0.16
50	-	-	-	-	-	70.00	21.00	8.30	1.70	0.55	0.20
55	-	-	-	-	-	85.00	26.50	9.70	2.20	0.65	0.24
60	-	-	-	-	-	100.00	32.00	11.50	2.50	0.75	0.28
65	-	-	-	-	-	-	37.00	14.00	2.90	0.90	0.34
70	-	-	-	-	-	-	42.00	16.30	3.50	1.08	0.40
75	-	-	-	-	-	-	48.00	18.50	3.90	1.20	0.46
80	-	-	-	-	-	-	55.00	22.00	4.30	1.40	0.52
85	-	-	-	-	-	-	60.00	23.00	5.00	1.50	0.58
90	-	-	-	-	-	-	68.00	27.00	5.60	1.70	0.65
95	-	-	-	-	-	-	75.00	30.00	6.30	1.90	0.72
100	-	-	-	-	-	-	85.00	33.00	7.00	2.20	0.80

SESSION IV (B)

DESIGN OF

TREATMENT AND STORAGE SYSTEMS

Design of Treatment and Storage Systems:

As has already been discussed earlier Unit operations for water treatment treatment required would be dependant on the quality of the raw water. However the plantations may be expected to design systems that include the following:

1. Source
2. Sedimentation
3. Coagulation
4. Filtration
5. Aeration
6. Chlorination
7. Storage

Although the treatment would depend on the use eg., factory, domestic etc., it may be safer to design the system to include all the above stages. It may also be stated here that although several sophisticated methods are available the simple designs existing in some plantations and suitable for most others have been suggested. The schematic diagram of Treatment plants is shown in figure 34 and detailed designs/descriptions are treated separately later. It may be noted that all the chambers should be of concrete (the thickness of the walls varying between 0.22 to 0.3 metres). Such concrete chambers would help reduce recurring cost.

Plain Sedimentation Chamber: Plain sedimentation is useful for the removal of gross turbidity and for reduction of bacteria especially pathogenic bacteria. For highly turbid water it is necessary to reduce the turbidity to a certain extent before allowing it to pass through the flocculation and filtration process. If this is not done, the dosage and time required for flocculation will have to be high and the filtration plant will have to be of larger capacity.

The basic data required for the design of a plain sedimentation chamber are:

- The turbidity of the raw water measured for different times of the year.
- The sieve analysis data of the suspended particles.

A – RAW WATER INFLOW
B – PLAIN SEDIMENTATION CHAMBER
C – COAGULANT FEEDER
D – COAGULATION (MIXING & SETTLING) CHAMBER
E – SLOW SAND FILTER
F – AERATION CHAMBER
G – CHLORINE FEEDER
H – STORAGE/DISTRIBUTION CHAMBER
I – TREATED WATER OUTFLOW TO DISTRIBUTION

STATIC HEADS

h_1 – WATERLEVEL IN B–CENTRE OF PIPE AT THE BYPASS INTO COAGULANT FEEDER.
h_2 – CENTRE OF PIPE AT BYPASS INTO COAGULANT FEEDER – W.L. IN D = [illegible] m
h_3 – W.L. IN D – W.L. IN E [illegible]
h_4 – W.L. IN E–CENTRE OF DISCHARGING END OF MANIFOLD [illegible]
h_5 – W.L. IN F–CENTRE OF PIPE AT BYPASS INTO CHLORINE FEEDER.
h_6 – CENTRE OF PIPE @ BYPASS INTO CHLORINE FEEDER – W.L. IN [illegible]

FIG. 4 SCHEMATIC DIAGRAM OF A TREATMENT PLANT

Plain sedimentation is a process in which the suspended particles are allowed to settle down by the virtue of gravity. The time taken for settling down is propotional to:

- The relative density of the particles
- The sum of forces acting upon the particles

For a given particle the rate of settling down will be faster if the sum of resultant forces acting on it perpendicular and opposite to the direction of gravitational force is minimum or nil. This will be facilitated by allowing the raw water to pass through a long chamber before cascading. Velocity breakers are provided in the chamber. The following Table XI gives the time taken by different kinds of particles to settle down in still water.

TABLE XI

Material	Diameter(m.m.)	Rate of settlement(metres/hour)
Coarse sand	1.00 - 0.5	366 - 194
Fine sand	0.25 - 0.1	100 - 29
Silt	0.05 - 0.005	10 - 0.14
Fine clay	0.001 - 0.0001	0.005 - 0.0005

It can be seen that plain sedimentation is not effective if the turbidity of raw water is caused by clay or silt. The design of the plain sedimentation chamber takes the following factors into consideration:

(i) The rate of inflow of raw water

(ii) The turbidity of raw water and the content of coarse and fine sand.

The velocity of flow in the chamber should not exceed 0.15m per minute.

The width of the chamber is given by:

$W \times D \times V = Q$

$$W = \frac{Q}{D \times V} = \frac{Q \times 60\ V}{2 \times 0.15} = 200\ Q$$

where: W = width of chamber in metres

D = water depth = 2m

V = velocity in M/sec = $\frac{0.15}{60}$

The length of the chamber is approximately given by:

$L = 2(1 + W)$

where: L = length in metres

If the total length exceeds 2 metres then velocity breakers are provided at an interval of 2 metres. The outlet pipe is kept 0.3 m below the water-level. The diameter of this pipe varies from 100 mm to 150 mm depending upon the discharge.

Figure 35 represents the details of a plain sedimentation chamber.

Coagulation.

Coagulation is the final state of sedimentation by which about 80% of the suspended particles are separated and settled. Coagulants are added at a constant rate to the plain sedimentated water. The process in the simplest form involves (a) Coagulant feeding and (b) mixing and settling. The most widely used coagulant is Aluminium sulphate (Alum). The dosage varies depending upon the turbidity.

(a) Feeding: There are two methods of coagulant feeding viz., solution feeding and dry feeding. The solution feeding alum is dissolved in water before feeding at a constant rate to the raw water. In dry feeding the alum is added to the raw water in the form of powder or granules.

(b) Mixing and Settling: Mixing is required to facilitate maximum dissolution of the coagulant in water. Though there are several mechanical devices for mixing, the simplest method would be by letting the water pass through constrictions under high velocity. Water containing coagulants is allowed to pass through a rectangular basin with baffles (sheets of concrete) placed across the basin, one end of the alternate baffles being kept slightly apart from the opposite wall.

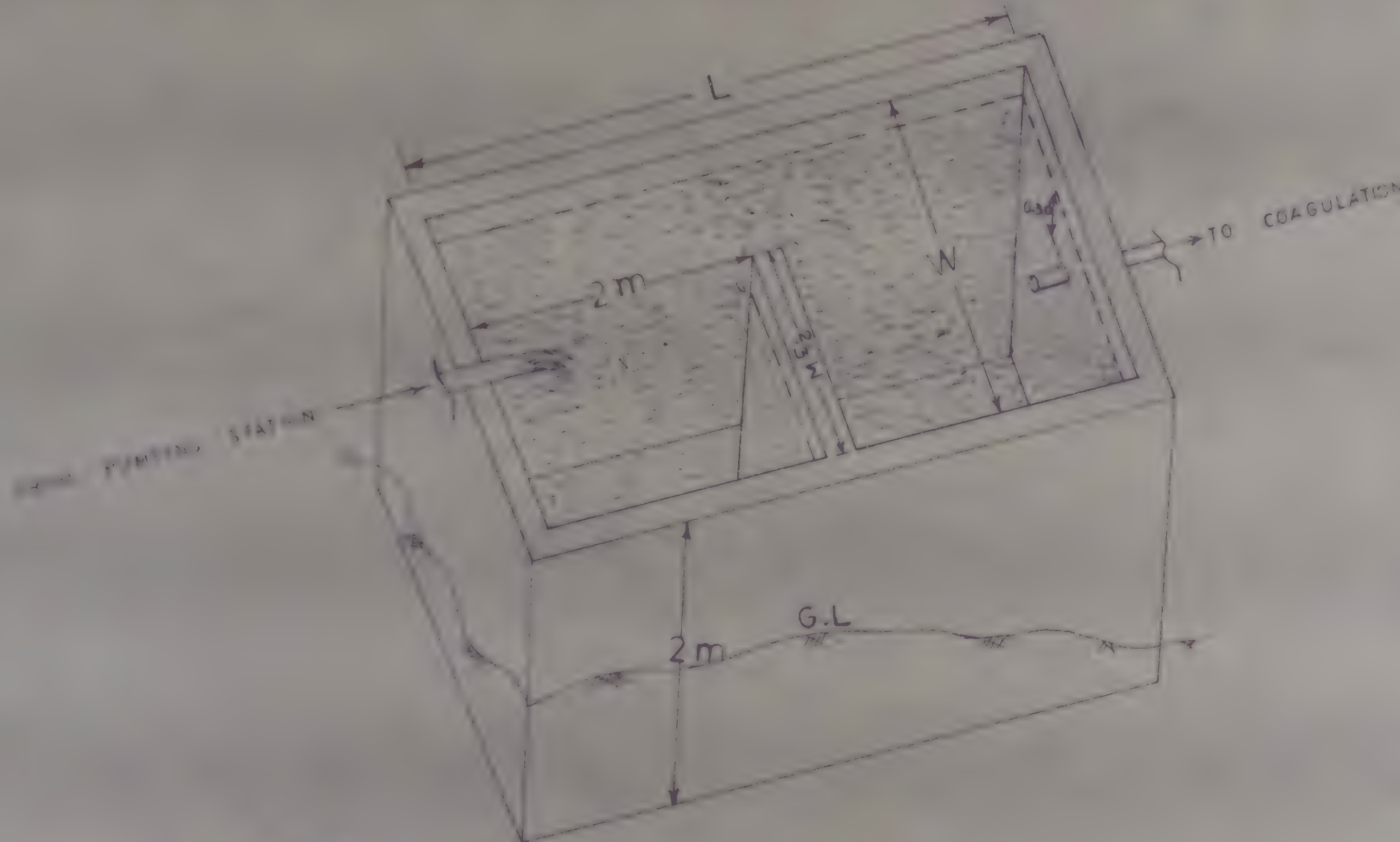


FIG. 3.5 PLAIN SEDIMENTATION CHAMBER

The approximate dimensions of a mixing and settling chamber for a treatment capacity of 1.5 lakhs litres per day is given in figure 36.

Slow Sand Filtration:

Filtration not only removes the finer suspended particles but also reduces the bacterial contents by 90 - 95 %. The best suitable and simple method of filtration for small water supply schemes is slow sand filtration. This method is most practical in treatment of water under the gravity system of water supply.

Design: (see figure 37)

(a) Raw Water: Turbidity less than 50 ppm for the water applied to the filter; higher turbidities can be filtered for short periods. Average turbidity should be less than 30 ppm for economical and continuous operation.

(b) Rate of filtration: This should be about 4 - 5 m^3/day per square metre of filter bed depending on the requirements. The lower the rate of filtration the more effective will be the treatment.

(c) Filter sand: Should be uniform, free of organic matter, and effective size should be in the range of 0.5 to 1 mm. The finer the filter sand; the more efficient will be its filtering action but the quicker it will clog, thus increasing operation costs. Good results have been obtained with somewhat coarser sand and higher loadings. This reduces the filter sizes and operation cost. With generally clear waters, free of pollution, higher raters are well justified.

(d) Filter bed: About 0.5 m of sand supported on 0.5m of graded gravel and pebbles of 10 mm and 15 mm respectively. Stones are packed near the outlet pipes to a height of 0.25 metres and a width of 1 metre from the outlets. The depth of water over the filter can be from 0.25 to 0.5 metres. Outlet pipes are spaced at a distance of 1.0 m but converge into a single manifold before being conveyed to the Aeration chamber.

Aeration:

Aeration is a process in which raw or partially treated water is exposed to the atmospheric oxygen for sufficient durations to remove unwanted

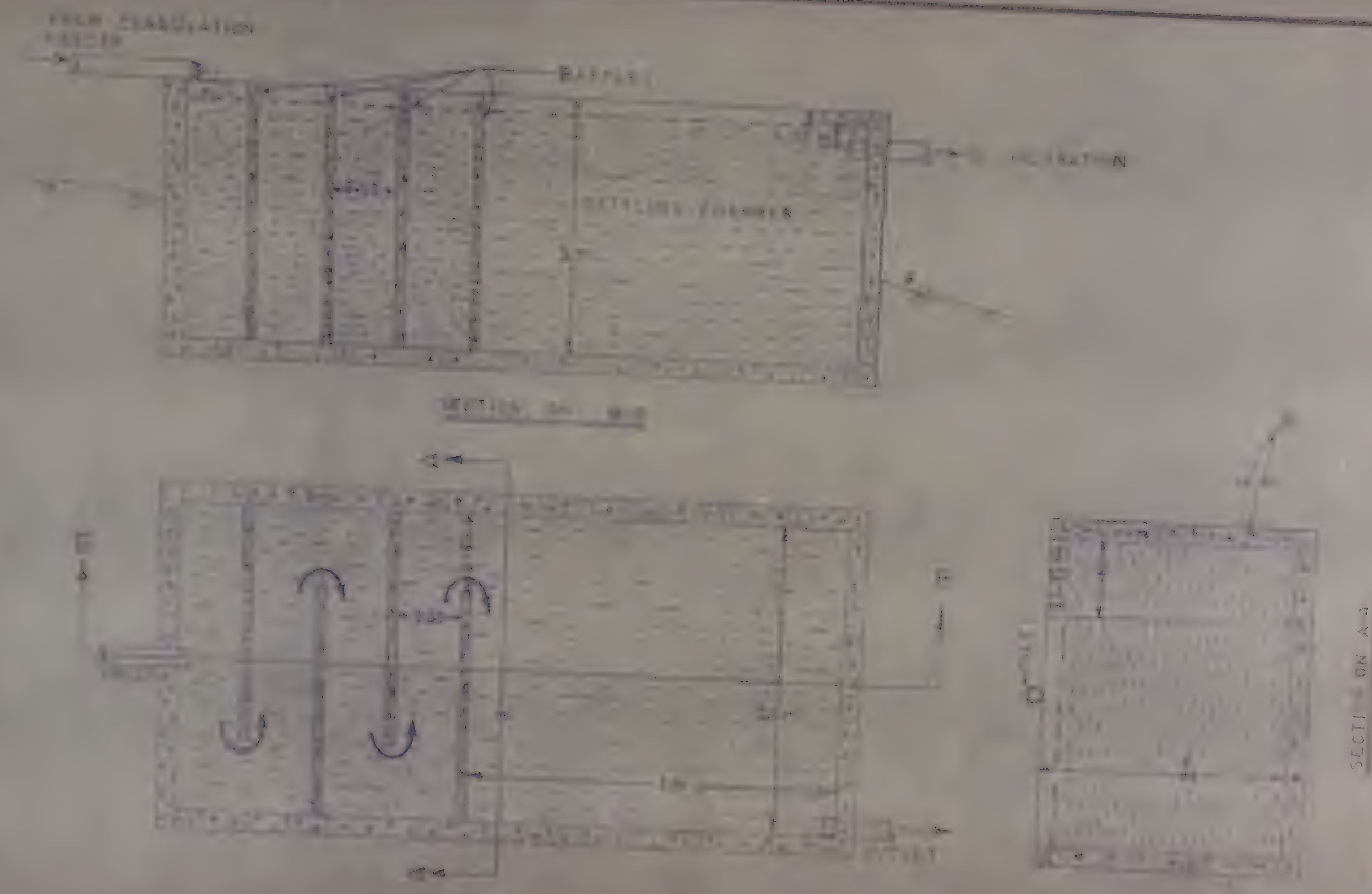


FIG. 6 COAGULATION CHAMBER (MIXING & SETTLING)

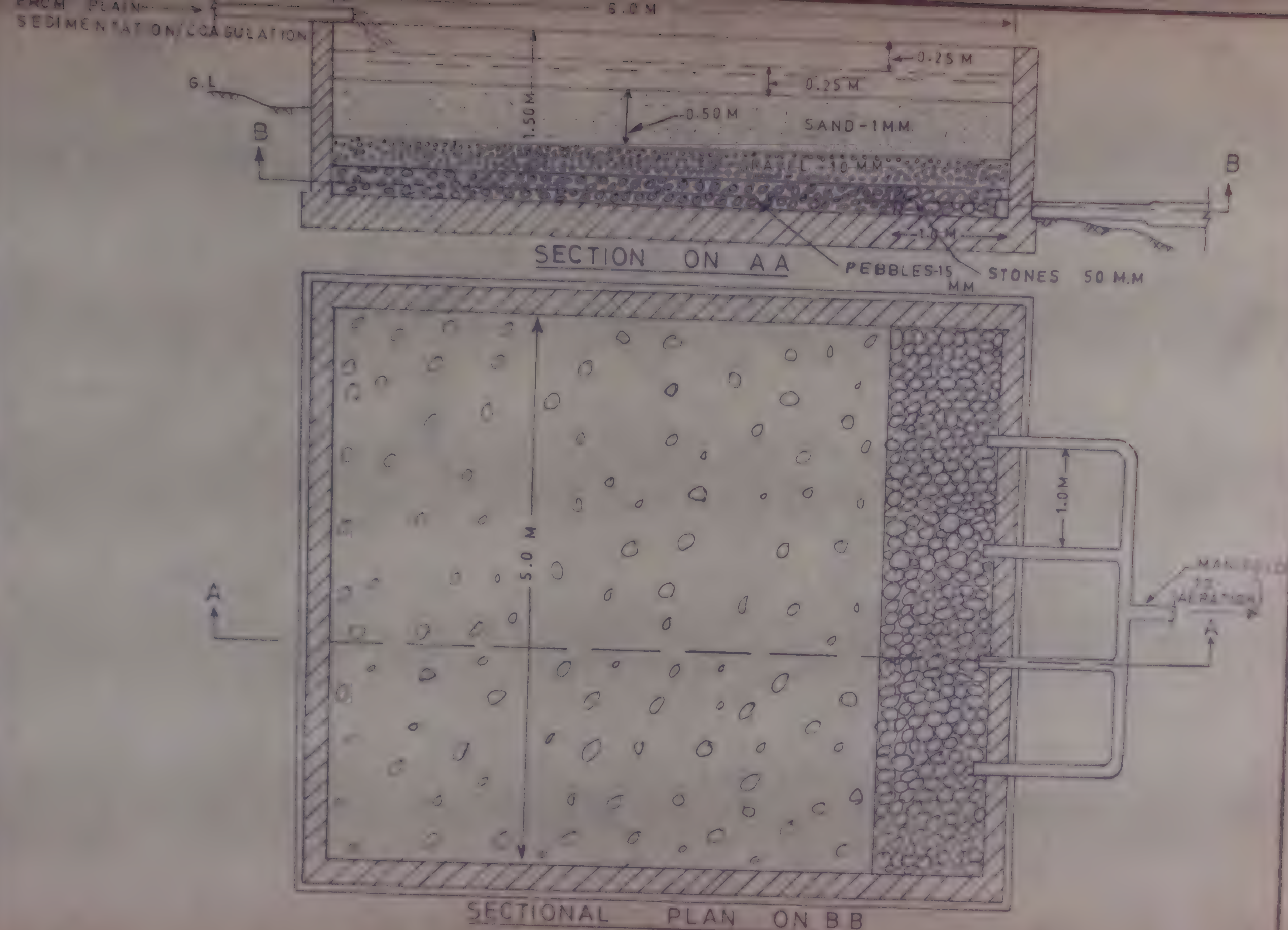


FIG.37 SLOW SAND FILTER

gases, iron, etc., thus improving the odour and taste. In addition, staining of plumbing fixtures is reduced. The simplest method for plantations involves allowing the water to flow in a thin sheet (10 - 20 mm) over a channel 0,5 m wide and 3 m long, the bed slope being 1 in 100. The figure 38 represents the details of such an arrangement. The aerated water is then taken to the chlorination plant before being conveyed to the storage chamber.

Storage Chamber:

The chlorinated water enters the storage chamber where the reaction of chlorine on the bacterial organisms is completed. The major aspects to be considered in the design of a storage chamber or distribution chamber are:

(a) The capacity

(b) The water level to be maintained.

(a) The capacity of a distribution chamber depends on the rate of pumping, frequencies and duration of power failure, pumping unit being rendered inoperative due to break downs, and the maximum consumption expected during non-pumping hours. As a rule of thumb, the capacity of such a chamber should be equal to one day's consumption.

(b) The relative water level to be maintained in the storage chamber with respect to other points like source, treatment plant, consumer points etc., depends upon :

- the level of the highest consumer points in the command of that storage chamber.
- the minimum head difference (pressure) required for the chlorine feeder to operate.

Considering the above factors the water level should be kept:

(i) 1/100th of the horizontal distance between the chamber and the farthest consumer point, above the level of the latter.

(ii) should be at least 1 m below the point on the inlet pipe from which the feeder pipe for chlorination has been taken. This point would be so selected on the inlet pipe coming from the aeration chamber

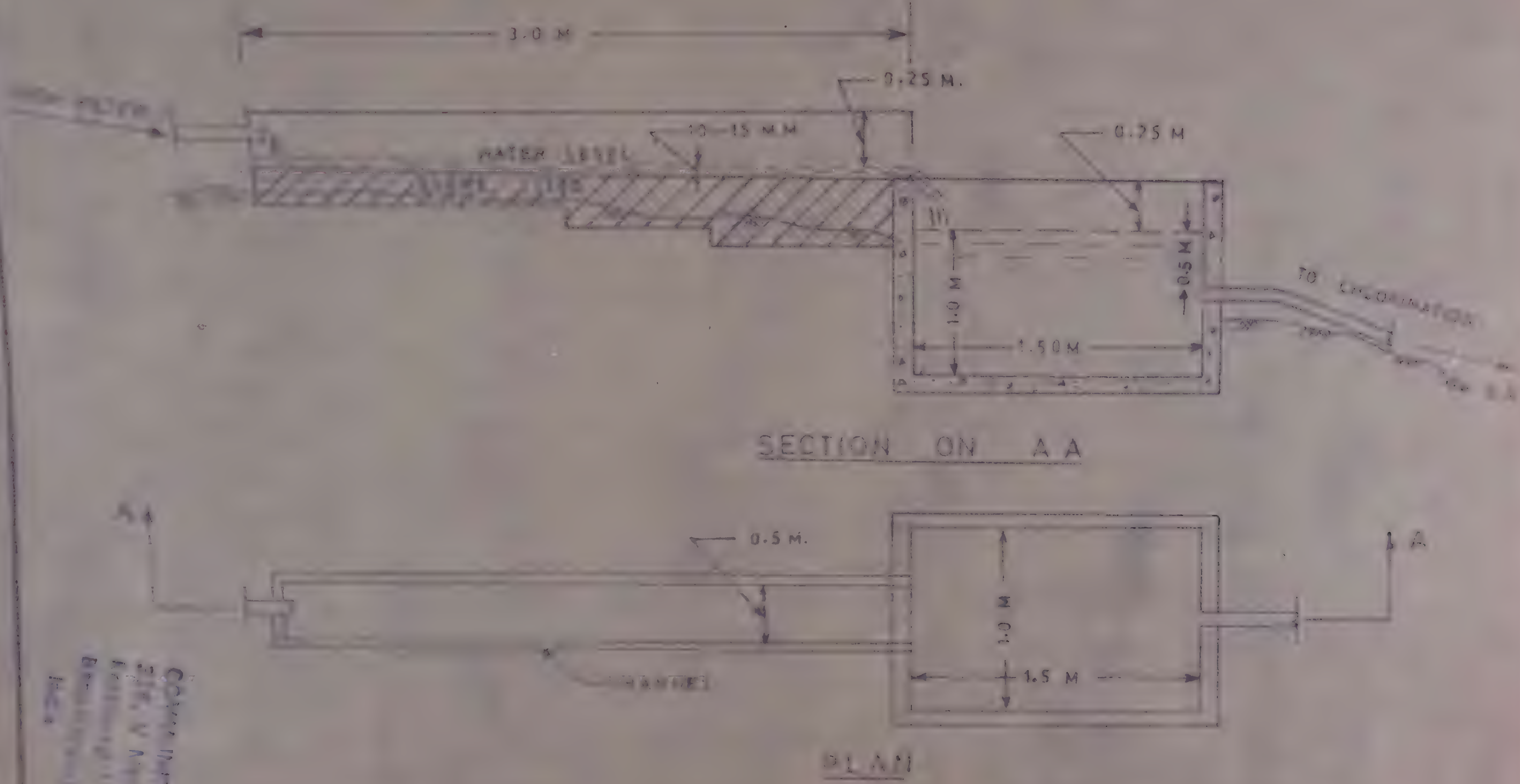


FIG.10 DETAILS OF AERATION CHAMBER

so that the difference between the water level in the aeration chamber and the centre of the pipe should be equal to the minimum head required for the chlorination plant to function.

It is always advisable to divide the storage chamber into two compartments in order to facilitate periodical cleaning. The details of pipe connections and other details are shown in figure 39. A wire mesh cover should be provided over the chamber to keep out birds, leaves etc., at the same time permitting ventil ation.

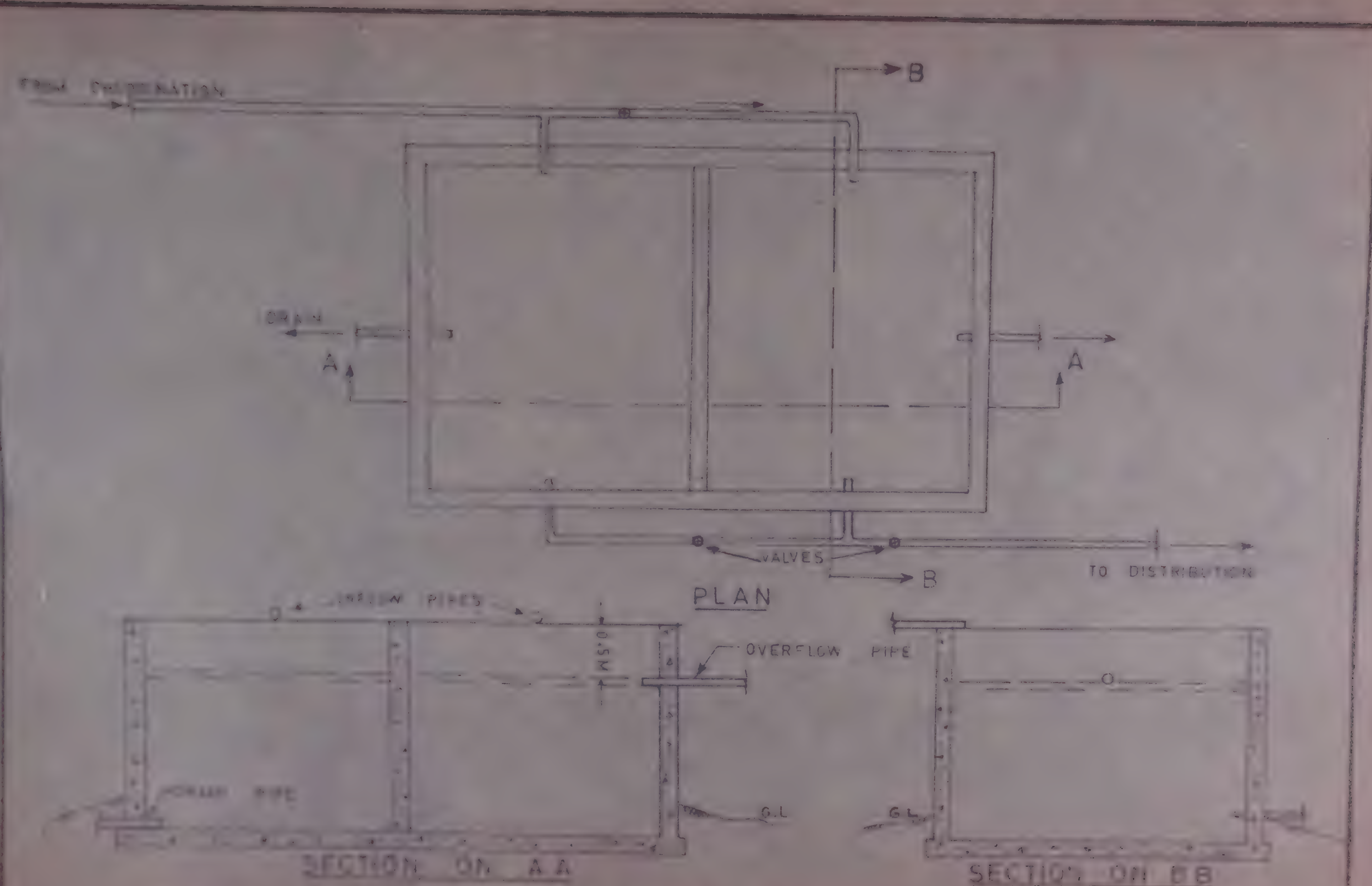


FIG. 37 DETAILS OF A STORAGE/DISTRIBUTION CHAMBER

SESSION VA

CONSERVATION
OF
WATER
IN
PLANTATIONS

Need for Conservation of Water:

In several estates the scarcity of drinking water is very acute during most months, whereas torrential flow may be seen in streams during the monsoons. There are means, though expensive to tap and store this excess water to be used during lean periods. Apart from this, the conditions can also be improved to a limited extent by controlled supply and economical use of water. It can also be generally said that the expenses involved are not beyond justification since the estates are essentially located where rainfall (basic source of water) is above average, hence avoiding the need to convey water from distances. The question is to optimize the use of the available sources. This question when considered for any scheme exploiting natural resources usually arises only when there is an excessive depletion of sources and/or an increase in the demand. In case of water supply schemes in water scarce estates, the problems are related rather to the increase in demand, than the depletion of sources. In such estates the need for conservation is imperative not only to meet the present demand but also to narrow down the ever-increasing demand supply gap.

Conservation Measures:

Conservation measures can be broadly classified into two categories:

- To impound/store/retain the excess water when available for use in lean periods with the help of suitable structures.
- Economic and controlled use of water.

Storage of excess water: Though there are various methods, like small dams, deries of check dams etc., to impound and store run off they may not be practical in plantations for the following reasons:

- They occupy large areas.
- The evaporation losses are high

The method hence suggested is detailed below:

A point is selected within the estate such that:

- it is higher than the highest inhabitations.
- it is in the shape of a small plateau.
- no rock is expected at least up to 4m below ground level.
- is nearer to the stream which has a torrential flow during monsoons.

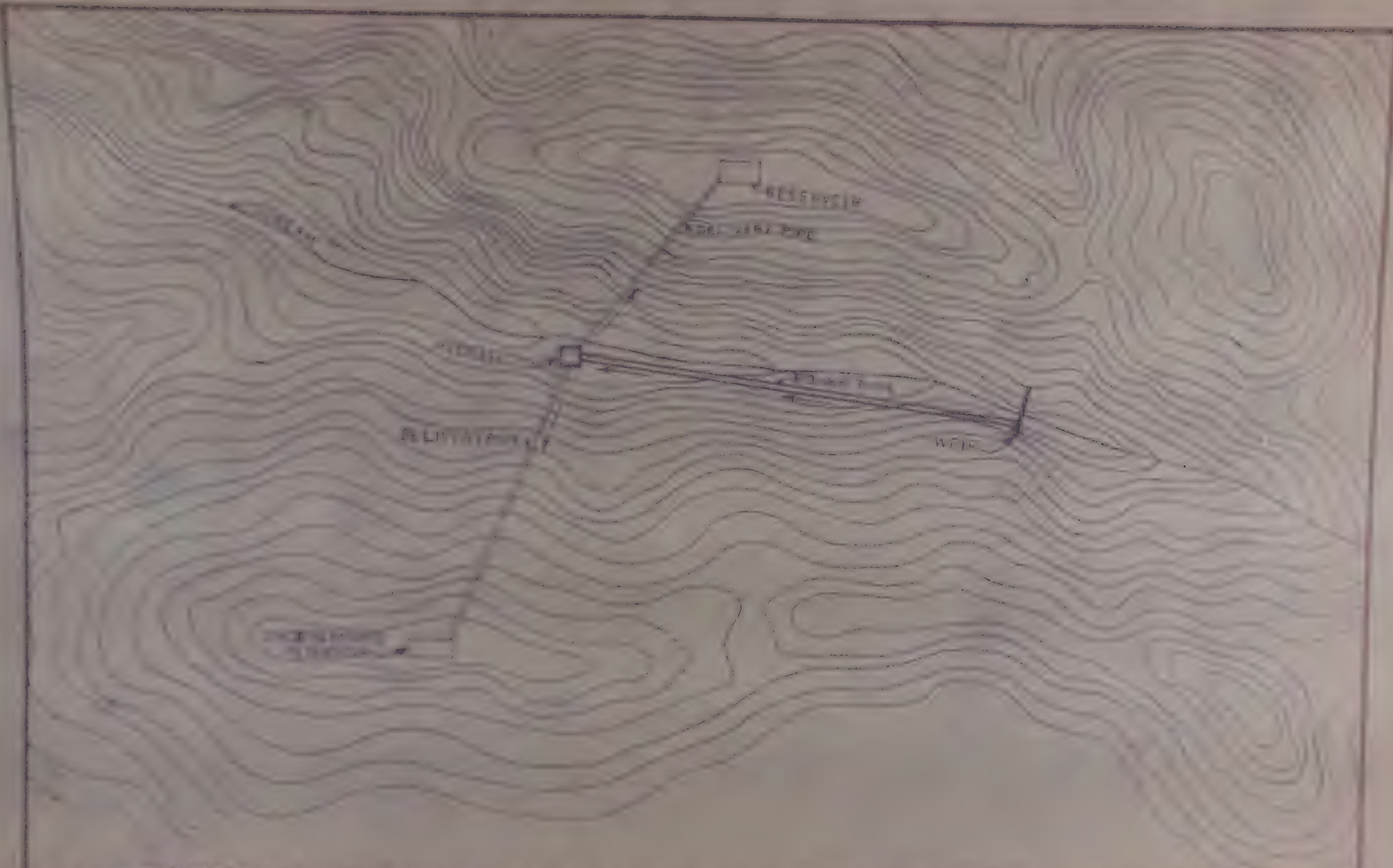
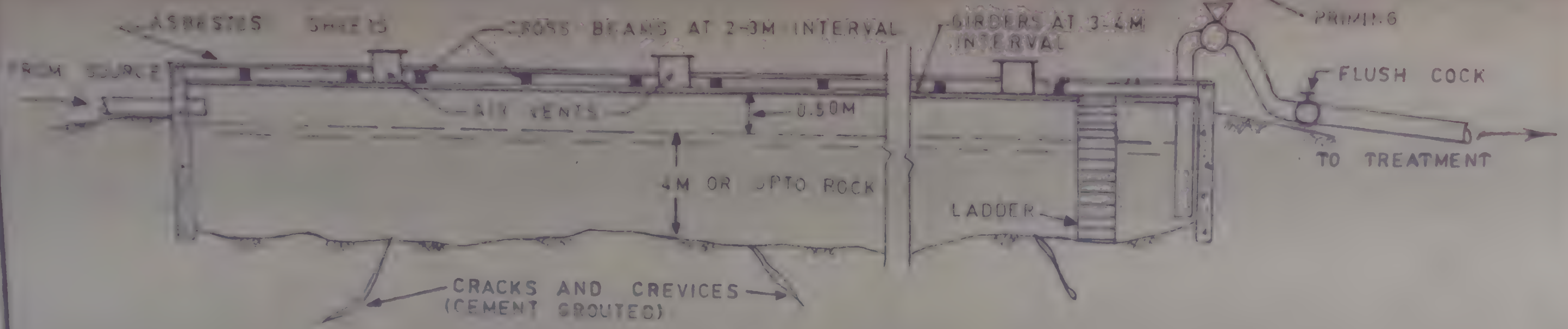


FIG. 40 TAPPING & STORING EXCESS WATER OF MOUNTAIN STREAMS.



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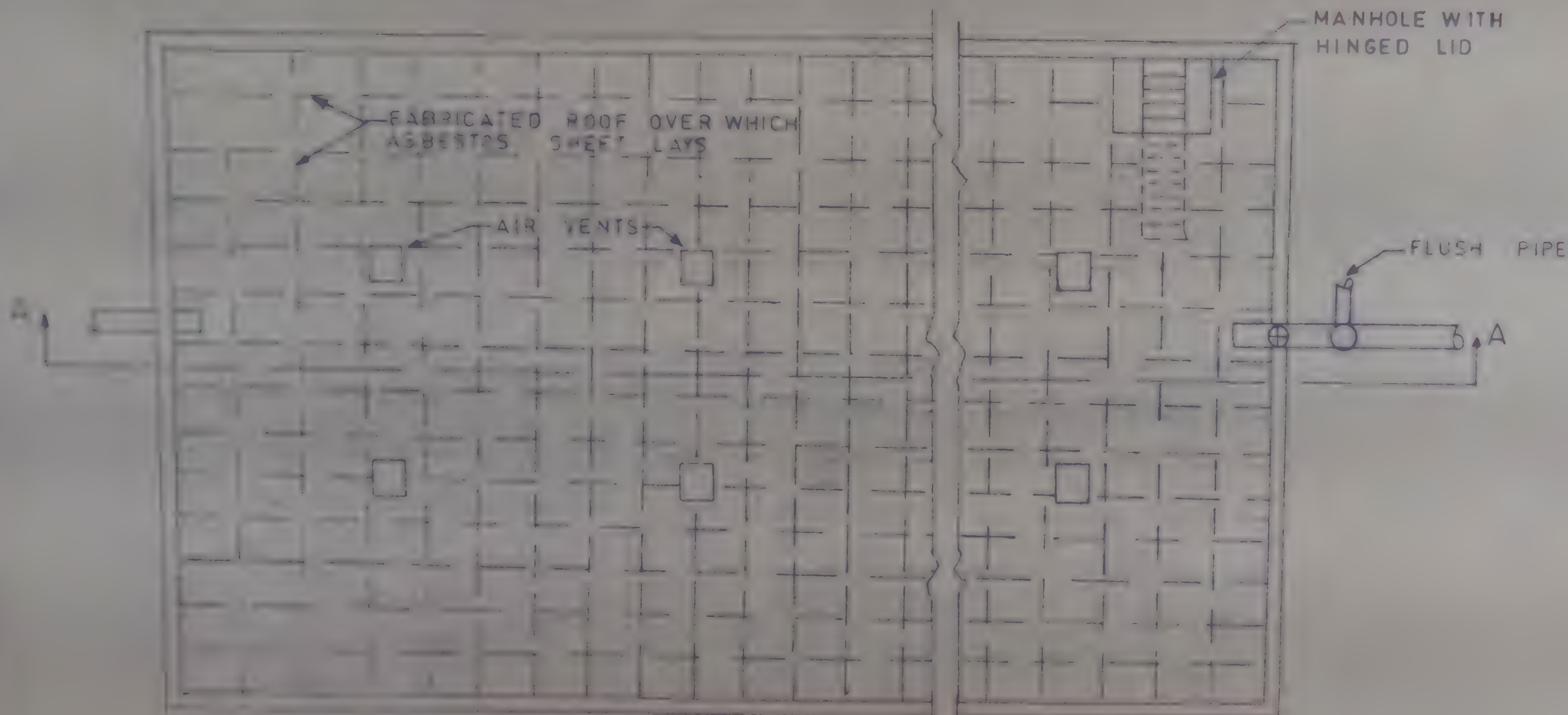

PLAN

FIG 4 UNDERGROUND RESERVOIR

However this size should not exceed 20m x 15m x 6m, though the number of tanks that can be provided maybe increased depending on the storage requirement. The design of the walls, the roof fabrications etc., is done only when the volume and the size of the tank is known.

A point to be stressed at this juncture is that the method of storing excess water briefed above should be adopted only as a last resort as heavy investments and large space are required for the storage tanks. The method however can be considered when despite pooling up all the possible sources, the supply demand gap and the drought frequency is very high. This could be favourably considered for protection as an insurance against failure of timely rainfall which may lead to crop failures. This will however be useful for 1 or 2 irrigations and could specifically be used to save young plants.

Controlled and Economic Use of Water:

In community water supply schemes experience shows that a considerable amount of water is wasted due to the negligence of consumers. Running taps and leaky connections are not uncommon scenes. The authorities have tried to solve the problem by restricting the supplies to a limited time. This has not only failed to curb waste but also added to the miseries of the consumers; long queues resulting in commotion. More often than not the taps run when water is not required or utilised. Such wastes must be avoided in scarce areas where water is supplied at high costs. The following guidelines are suggested to meet such problems.

a. All the public taps must be of spring loaded push open type.
b. No restriction should be laid on the supply duration. A round the clock supply is the best.
c. The consumer must be reminded to be restrictive in the use of water and to restrain children from unnecessary interferance with taps and connections.
d. One person for each labour line could be entrusted with the responsibility of inspecting and rectifying any defects in the taps and connections

e. Each group of taps (serving 6 families) each should have a separate water meter. This will help on locating and isolating the point of excess use or wastes if any.

SESSION V B

MAINTAINANCE OF WATER SUPPLY SYSTEMS IN PLANTATIONS

Need for Maintainance:

The aim of water supply system is obviously to make available good quality water with sufficient quantities based on available resources and requirements. There is hence a need for maintainence of the system so as to ensure that this aim is fulfilled. The detailed preventive maintainence required has been given under the check list ~~under 4A.2~~. these may be classified as follows:

(a) Sources: Springs
Wells: Open and Bored
Weirs
Collector/Jack wells
Direct Lift from river/stream

(b) Pumping sets.

(c) Quality

(d) Treatment Plant

(e) Conveyance and supply network

(f) Conservation structures.

CHECK LIST FOR MAINTAINANCE OF WATER SUPPLY SYSTEMS IN PLANTATIONS:

Springs

1. The area around should be kept clear and free from encroachment from animals, dead leaves etc., by constant checks.
2. The storage chamber should be cleaned periodically say bi-annually.
3. Periodic 6-monthly checks should be carried out for bacteriological and chemical (Fe) contamination.
4. A constant observation of the water level in a spring chamber should be maintained.

Well (Open):

1. The well should be desilted at least once a year.
2. Surface drains to be suitably diverted regularly.
3. Monitoring regularly for bacteriological and chemical pollutants at least 6 monthly
4. Well lining to be checked and repaired when necessary.
5. Periodic checks on water levels.

..2/-

Borewell:

1. The borewell should be desilted or cleaned after periodic checking for siltation. This can be done by flushing with a compressor.
2. Surface drains to be diverted.
3. Monitoring for bacteriological and chemical contamination at least 6 monthly.
4. Periodic checks on water level.

Weirs:

1. Annual check on the silt accumulation on the upstream side. Desilting if the depth of the accumulated silt exceeds 1/5th of the depth of weirs.
2. Periodic check to see any scouring of the wall or foundation.

Jack Wells/Collector Wells:

1. Annual de-silting of well.
2. The gravel packing around the well is likely to get clogged frequently in which case this may have to be replaced by clean coarser gravel.
3. If the slotted pipe gets clogged, this may also need gravel packing around it.

Pumps:

1. The turbidity of water should not exceed that prescribed for each pump. In case it does the dealer should be consulted to ascertain the special precautionary measures.
2. The pressure at the pump should not exceed the prescribed limit.
3. Servicing should be done without fail at the prescribed interval.

Conveyance:

1. Monthly inspection of the pipe lines and valves to detect any leakage.
2. Immediate rectifying.
3. An increase in pressure at the pump denotes some defect in the conveyance system.

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Treatment Plant (General)

1. The entry into premises in which the treatment plants and storage chambers are located should be restricted to authorised persons. Wire fences maybe erected if necessary.
2. These premises should be kept clean. Constant vigil is required to ensure such cleanliness.

Plain Sedimentation:

1. Periodical sampling of raw water and analysis for turbidity.
2. Annual cleaning.

Coagulation Chamber:

1. Periodical sampling of water that has undergone sedimentation to analyse for turbidity and decide coagulant dosage.
2. Checking the function of the coagulant feeder.
3. Annual cleaning.

Slow Sand Filter:

1. Check water level constantly.
2. An increase in water level shows that the top layers of filter bed are clogged.
3. When the clogging is excessive, causing the filter to overflow; the top 5 to 6 cm layer of sand is carefully removed, washed properly and installed back into place.
4. The sand bed should not be stirred.
5. Every two or three years preferably after the monsoons, when the raw water is comparitively clearer the entire filter materials should be removed carefully, washed and re-installed.

Chlorination:

1. The chlorine feeder should be inspected daily to ensure smooth functioning.
2. The dosage as determined by periodical analysis should be strictly adherred to.

Storage Chamber:

1. The pumping should be controlled so that the water level in the chamber does not exceed the overflow level. This may be known from experience.
2. Cleaning at least bi-annually.

Underground Reservoir :

1. Annual flushing using the flush cock.
2. This should preferably be done after the monsoons.
3. Annual inspection of the walls and the floor to detect leaks.
4. Immediate sealing of such leaks with cement grouting.

Public Taps:

1. Taps should be leak proof and immediately replaced if defective.
2. The premises should be kept clean and well drained.

S E S S I O N VI

F I N A N C I A L I M P L I C A T I O N S

O F

W A T E R S U P P L Y

I N

P L A N T A T I O N S

Financial Implications of Water Supply in Plantations:

Any investor would like all the implications in a scheme/project to be translated in terms of investments and returns, before considering the overall acceptability. For community water supply schemes, however the returns cannot be worked out directly in terms of increase in production. It is obvious that only a healthy working community can maintain a high production level. Specifically referring to plantations, reports of high incidence of water-borne diseases has convinced managements that providing safe and sufficient drinking water at higher initial investments is of prime importance.

An attempt has been made to illustrate the method of costing a water supply scheme in plantations - this would be indicative more of the implications rather than actual cost which would vary greatly from each estate.

Basis for Working out Costs and its Limitations:

The typical estate referred to earlier, under assessment of water (figure 1) is considered for working out the costs. The schedule of rates for different works and items has been arrived at based on those obtained from various sources. However, for actual estimates, the rates prevailing in the area should be applied. The costs may be classified on the following lines:

(1) Capital Costs

(2) Recurring Costs

Capital Costs:

These include items such as:

(a) Project formulation

(b) Intake Structures

(c) Lifting Devices

(d) Conveyance and Delivery Systems

(e) Treatment and Storage

(f) Conservation Structures

Project formulation involves investigation, planning, designing and costing. The items under intake structures would be spring protection chambers, wells, weirs, etc., Under lifting devices would be shown pumping equipment and pump houses, whilst conveyance and delivery includes pipe, valves and

..2/-

associated fixtures. The chambers, equipment and materials required for sedimentation, coagulation, filtration, aeration, chlorination and storage fall under the head: Treatment and Storage. Conservation structures would include weirs, hydrams, pipes and underground storage chambers.

The Capital Costs under different heads are worked out as follows:

(a)	PROJECT FORMULATION (2,500 acres):			
(i)	Investigation/Evaluation @ Rs 70/- per acre		50,000	
(ii)	Planning, Designing and Costing @ Rs 10/- per acre		25,000	75,000
(b)	INTAKE STRUCTURES:			
(i)	Spring Protection Chamber	(1 No)	4,500	
(ii)	Jack/Collector Well	(1 No)	10,000	
(iii)	Openwell 10m x 6m, lining 6m	(1 No)	8,000	
(iv)	Borewell 20m depth with about 10m casing	(1 No)	11,000	33,500
(c)	LIFTING DEVICES			
(i)	Reciprocating/Centrifugal Pump and motor for Spring, Openwell, River lift and Jackwell	(4 Nos)	60,000	
(ii)	Submersible pump for bore well and accessories	(1 No)	20,000	
(iii)	Pump houses	(4 Nos)	20,000	1,00,000
(d)	CONVEYANCE AND DELIVERY:			
(i)	Pressure pipes, 3420 metres @ Rs 120/- per metre(HDPE)		4,10,400	
(ii)	Gravity pipes, 4370 metres @ Rs 100/- per metre(HDPE)		4,37,000	
(iii)	Auxillary fitting like valves @ Rs 5/- per metre of pipeline		38,950	8,86,350

..3/-

(e) TREATMENT AND STORAGE:

(i)	Sedimentation Chamber	(3 Nos)	8,500	
(ii)	Coagulant Feeder	(2 Nos)	5,000	
(iii)	Slow Sand Filter	(3 Nos)	15,000	
(iv)	Coagulation Chamber	(2 Nos)	6,000	
(v)	Aeration Chamber	(3 Nos)	4,500	
(vi)	Chlorination Equipment	(5 Nos)	17,500	
(vii)	Storage Chamber, Large	(2 Nos)	25,000	
(viii)	Storage Chamber, small	(6 Nos)	24,000	1,05,500
				12,00,350

The above capital cost would be typical for an estate that can meet its requirement from available sources through the year.

In case of estates where available sources are inadequate conservation structures would require additional capital investment as detailed below (assuming that an Estate with 2000 population needs 50% of its requirements for 3 dry months in the year from conserved sources):

(F) CONSERVATION STRUCTURES:

(i)	Weirs	(2 Nos)	3,000	
(ii)	Hydrams	(2 Nos)	16,000	
(iii)	Pipelines for Hydram 3,000 metres @ Rs 150/- per metre(G.I. special quality)		4,50,000	
(iv)	Underground Storage Reservoirs	(3 Nos)	2,40,000	7,09,000
				19,09,350

Recurring Costs:

The cost of recurring and maintaining the water supply system installed or the recurring costs may be categorized as follows:

(a) Intake structure

(b) Lifting Devices

(c) Conveyance and Delivery System

(d) Treatment and Storage Chamber

..4/-

(e) Conservation Structures
(f) Interest on Capital Investment
(g) Personnel

(a)	INTAKE STRUCTURES:		
(i)	Depreciation on the capital assuming 20 year life period..	1,675	
(ii)	Maintaining (periodical repairs) 2% of Capital Investment..	670	2,345
(b)	LIFTING DEVICES:		
(i)	Depreciation (pumping equipment and accessories) assuming 10 year life period	8,000	
(ii)	Depreciation on pump houses assuming 20 year life period ..	1,000	
(iii)	Energy consumption ..	65,000	
(iv)	Maintainance @ 5% of cap.investment	5,000	79,000
(c)	CONVEYANCE AND DELIVERY SYSTEMS:		
(i)	Depreciation on capital investment assuming 20 year life period(HDPE)..	44,318	
(ii)	Maintainance @ 2% of capital investment	17,727	62,045
(d)	TREATMENT AND STORAGE CHAMBERS:		
(i)	Depreciation on capital investment on civil structures assuming 20 year life period	4,150	
(ii)	Depreciation on capital invested in equipment eg. Chlorinators and Coagulant feeders assuming 10 year life period	2,250	
(iii)	Maintainance @ 2% of capital investment	2,110	
(iv)	Cost of chemicals(Inclusive of potency loss in storage)	5,000	13,510

..5/-

(e) CONSERVATION STRUCTURES:

(i)	Depreciation on capital investment assuming a 20 year life period	35,450	
(ii)	Maintainance cost @ 2% on the Capital Investment	14,180	49,630

(f) INTEREST ON CAPITAL INVESTMENT @ 15% per annum

(i)	Project formulation	11,250	
(ii)	Intake Structures	5,025	
(iii)	Lifting Devices	15,000	
(iv)	Conveyance and Supply Systems	1,32,953	
(v)	Treatment and Storage Chambers	15,825	
(vi)	Conservation Structures	1,06,350	2,86,403

(g) PERSONNEL:

Salary benefits to 6 operators on 3 shifts @ Rs 300/- p.m.	21,600	21,600
Total		5,14,533

Cost Analysis:

An attempt has been made to illustrate the distribution of expenditure on various heads under water supplies. This is made with reference to the typical estate referred to earlier. This would be a guideline for allocation of finance in the planning stage.

$$\text{Cost/litre} = \frac{\text{Total Annual Recurring Cost}}{\text{Total Water Pumped Annually}} = \frac{5,14,533}{365 \times 1,46,600} = \text{Rs } 0.01$$

= 1 paise/litre

Head	Capital	Recurring
Cost per Capita (a) Management	Rs 2,368	730.00
(b) Labour	Rs 474	146.00
% Cost for Domestic Use	54.58	54.58
" " " Livestock	4.50	4.50
" " " Hospital	17.05	17.05
" " " Production (Factory)	23.87	23.87

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